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Mechanically Stabilized Earth (MSE) structures, true bridge abutment & GRS-IBS

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ABSTRACT

The salient features of MSE structures are its flexibility and ductility. These features have allowed MSE to be exploited for seismic applications. With the capability of MSE to mobilise to dissipate the seismic forces, it provides opportunity for the system to remain perpetually in the elastic state even in extreme seismic events. As such, MSE structures have transitioned from ensuring life safety to seismic resilience expectation. As MSE evolves from static applications to subsequent load bearing seismic applications, the design methodologies get complex. The modification and simplification of the design methodologies of MSE so as to assist practicing engineer to access this technology, is the presentation of this paper. The popularity of MSE has further given rise to Geosynthetic Reinforced Soil – Integrated Bridge System (GRS-IBS).

1 INTRODUCTION

MSE consists of facing elements, compacted soil backfill interlayered with reinforcement placed horizontally to form a gravity-retaining structure. The wall system relies on vertical stress of the weight of the reinforced-soil mass to resist lateral pressures from earth, surcharge and seismic thrust. As the wall system is flexible, it accommodates relatively large total and differential settlements without distress. Besides being cost-effective, MSE suits applications in regions of high seismicity.

2 DESIGN METHODOLOGY OF LATERAL EARTH PRESSURES

2.1 Coulomb (1776) and Rankine (1857) Earth Pressure Theories

The determination of lateral earth pressure of retaining wall is based on Coulomb (1776) or Rankine (1857) earth pressure theories.

Both theories essentially model the weight of the soil mass sliding along a theoretical plane of failure. The lateral earth pressure, Pa, is the net force required to hold the wedge of soil in place and satisfy equilibrium. The major difference between the two theories is that the Coulomb model and equations account for friction between the back of the wall and the soil mass as well as wall batter.

The friction at the back of the wall face and at the back of the reinforced zone for external stability computations provides an additional force component that helps support the unstable wedge of soil. Because of these additional resisting forces, the lateral earth pressure calculated by Coulomb is generally less than the earth pressure that would be predicted by the Rankine equations. There is always a vertical component Pav (=Pa x sin(δ-ι)) of the lateral earth pressure in Coulomb model.

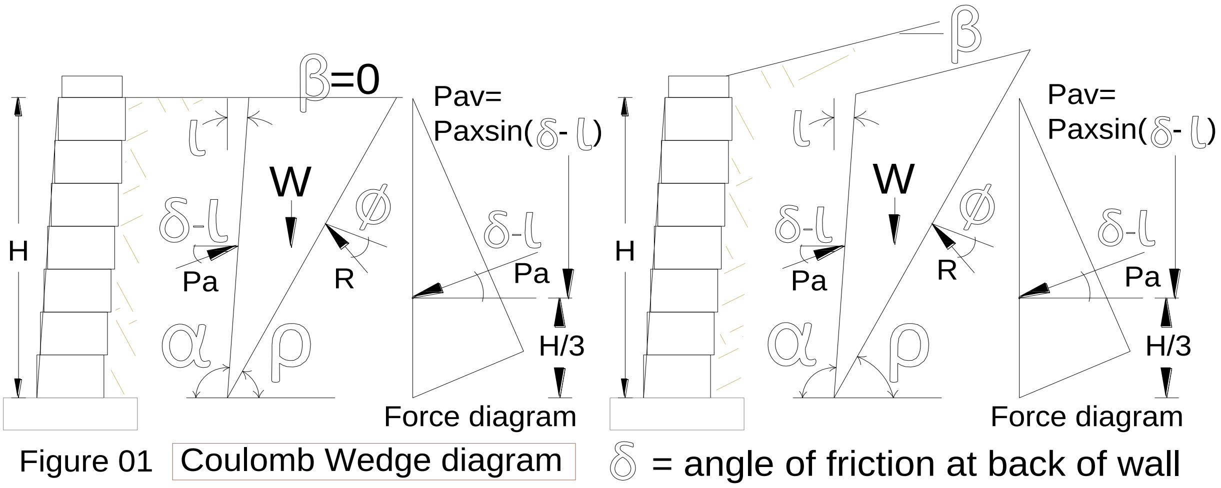


Figure 1: Coulomb wedge diagram

The advantage of using a Rankine earth pressure methodology is that no assumption has to be made with regard to friction between the wall structure and retained soil mass. Consequently, the Rankine theory provides simpler formula, as the failure plane definitions are easier to use and check.

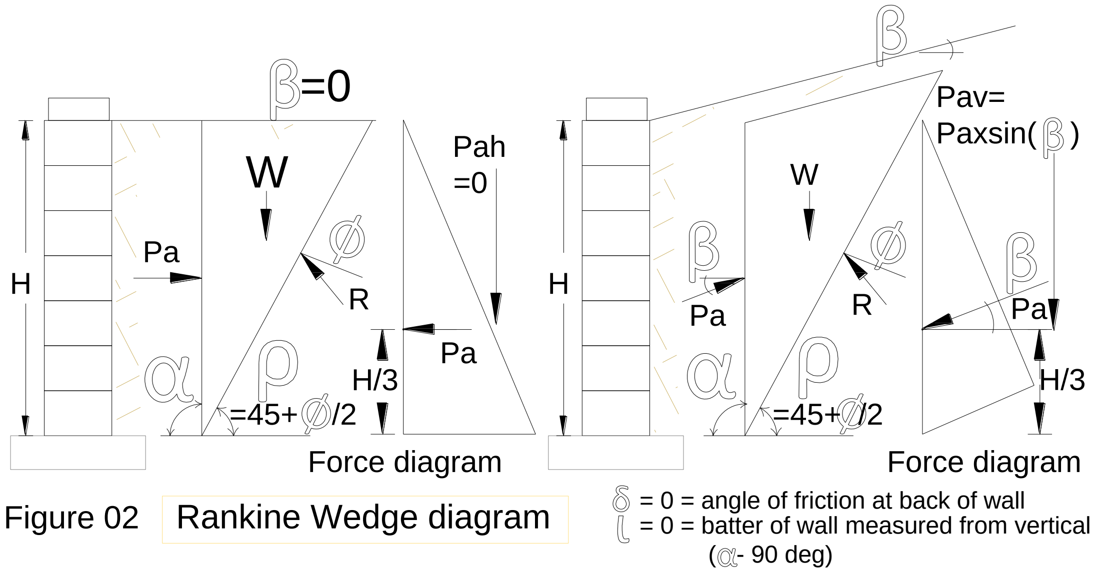


Figure 2: Rankine wedge diagram

As a result, Rankine design methodology is appealing in complex seismic analysis of MSE. American Association of State Highway and Transportation Officials (AASHTO) generally apply Rankine design methodology for MSE structures.

While the earth pressure envelope for rest (static) application is not influenced by the rigidity of the retaining system, the seismic earth pressure thrust (dynamic) envelope is greatly influenced by the rigidity and flexibility of the retaining wall system.

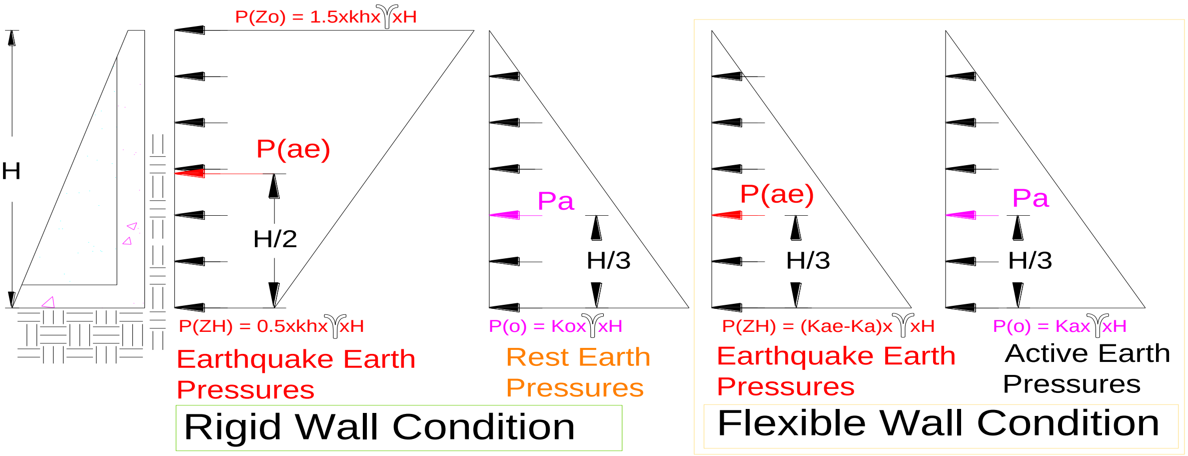


Figure 3: Influence of seismic earth pressure thrust on flexible wall

3 INTERNAL AND EXTERNAL ANALYSIS OF MSE

The analysis of MSE structures involves internal and external stability requirements. The internal stability analysis involves mainly tensile and pull out requirement of the reinforcement which is generally not a concern as the strength and spacing of the reinforcements can be varied to meet the requirement. As for external stability analysis, it involves safety against overturning around the toe, sliding on the base and bearing capacity of the foundation soil. Likewise the former two requirements are normally met with appropriate selection of the geometry of the MSE structures. Generally bearing capacity controlled the design of MSE structures. While flexibility is the salient feature of MSE structures which allows settlement to be tolerated, analysing to predict the settlement to comply with the AASHTO code which is generally limited to an allowable of 1% total differential settlement is a challenge. Numerical analysis is the appropriate tools. It is however complex which does not auger well with practicing engineers. As such, the analysis of MSE structures based on the combination of soil classical plasticity theories (Rankine) and empirical knowledge is appealing and is practised. Numerical analysis is employed for verification purposes.

A typical force diagram of MSE for static or at rest based on Rankine is as follows:

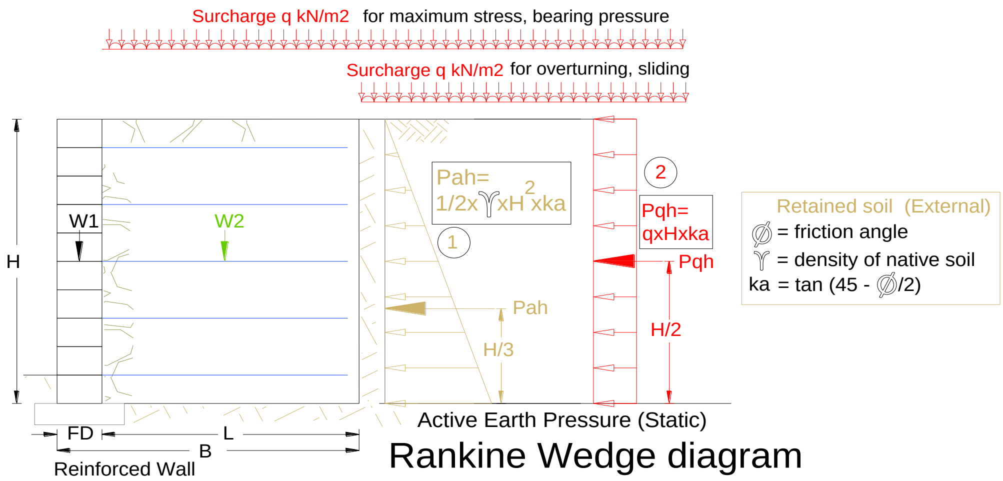


Figure 4: Rankine force diagram for MSE structure for static or at rest only

3.1 Dynamic (seismic) or pseudo-static forces

In the external stability analysis, the horizontal seismic acceleration induced two horizontal forces on the MSE structure namely:

- Dynamic inertia force (P_{ir}) proportional to the height of MSE and
- Dynamic earth thrust (P_{ae}) from retained soil

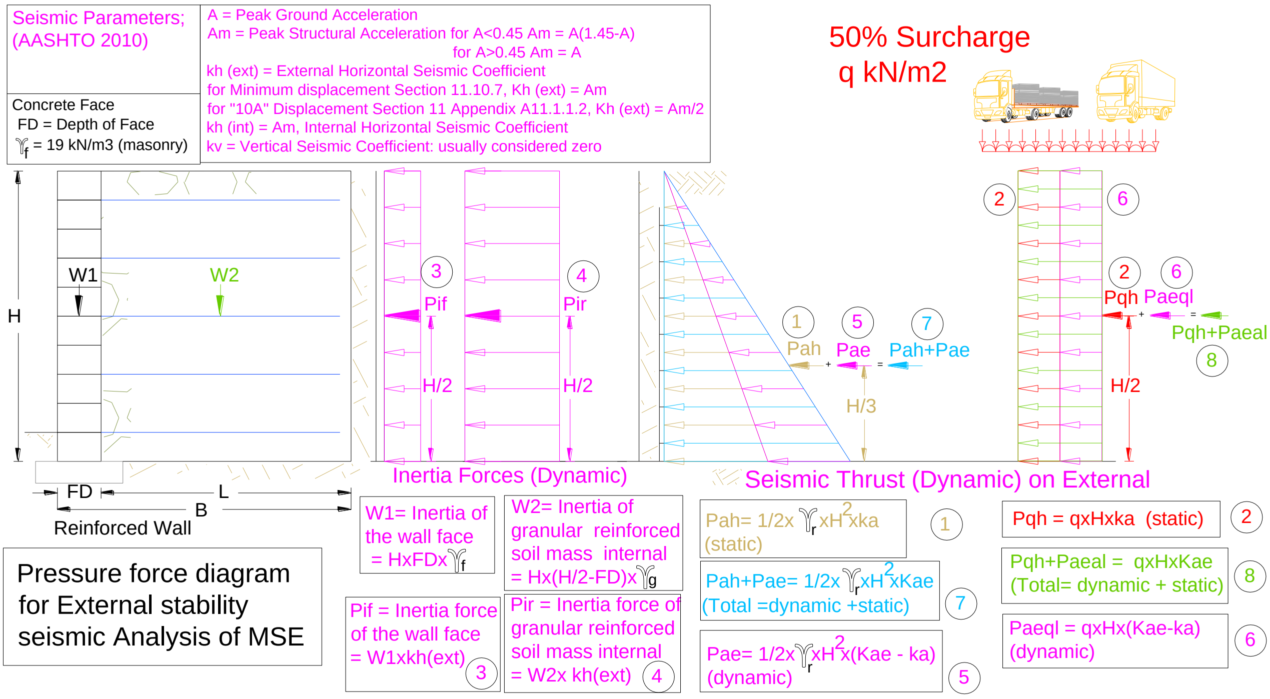


Figure 5: Rankine force diagram for MSE structure for the combination of static and seismic forces

With the forces resolved the force diagram shall be as follows:

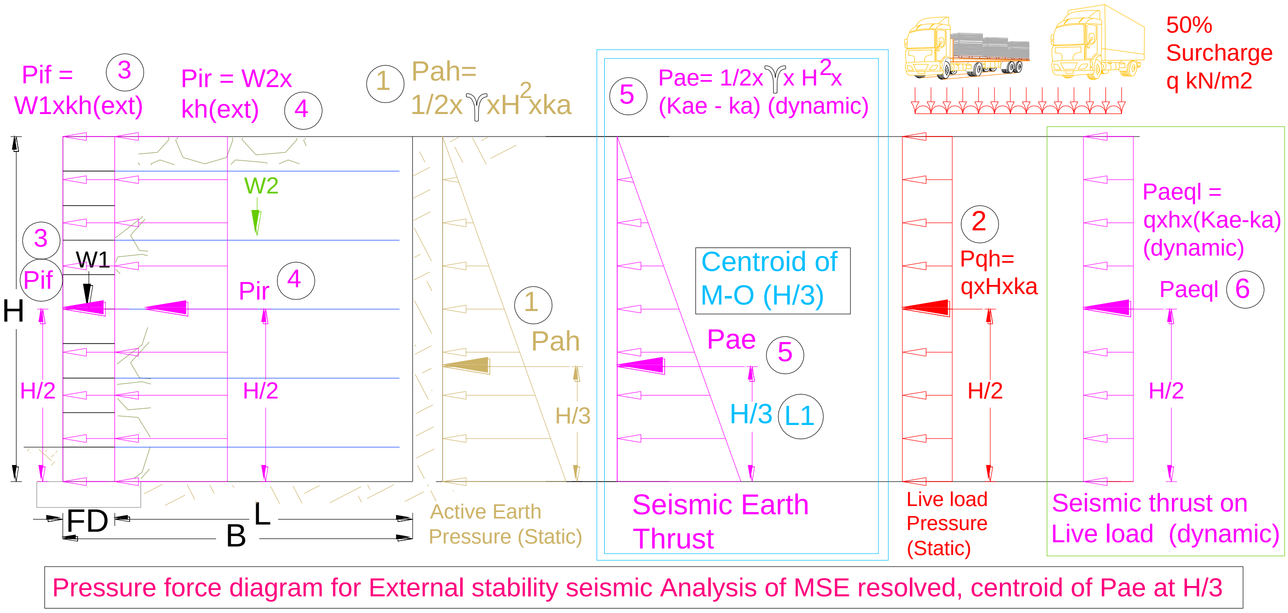


Figure 6: Rankine force diagram with pseudo-static forces resolved for centroid of Pae at H/3

3.2 Possible revision of location of centroid of Pae (seismic thrust)

While the AASHTO 2012 code does specify the vertical centroid to be H/3, it does recommend for higher centroid (0.6H) from the base for situation where the impact of failure is high.

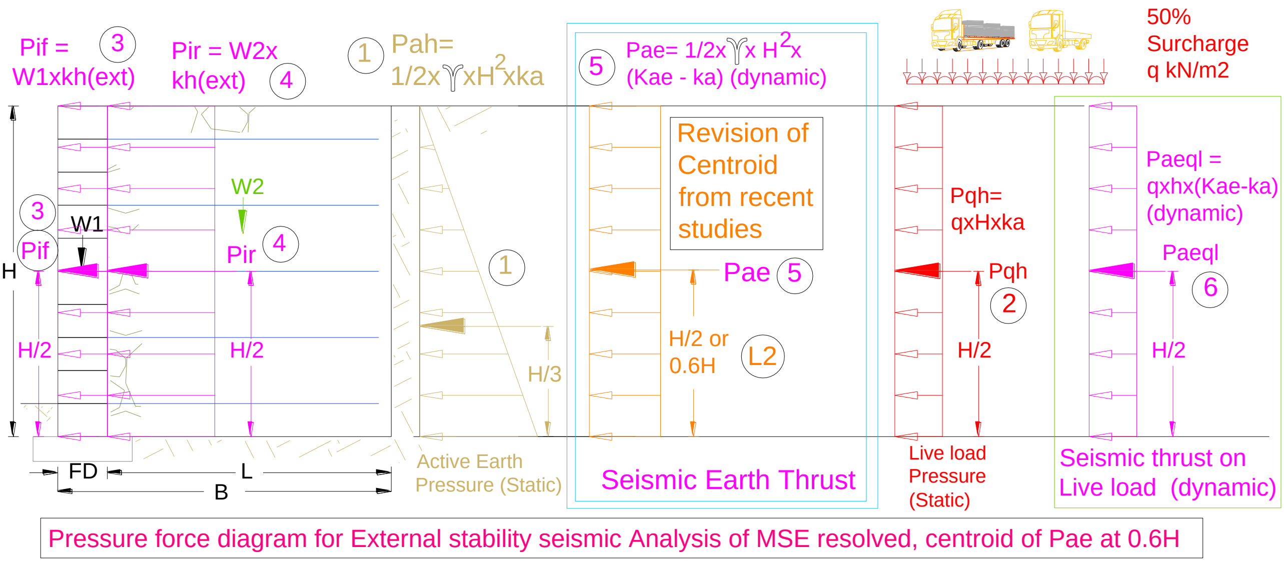


Figure 7: Rankine force diagram with pseudo-static forces resolved for revised centroid of Pae at 0.6H

The variation of the location of the vertical centroid is further highlighted by an article presented by Nicholas Sitar – Seismic Response of Slopes and Design of Retaining Structures, presented to New Zealand Geotechnical Society 2017.

3.3 The revision for the determination of coefficient of seismic/dynamic earth pressure

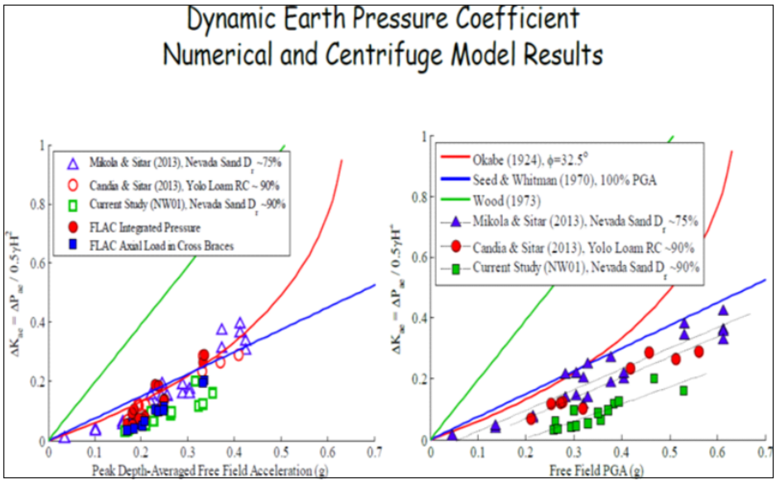
In the article presented, N. Sitar presentation highlighted various research studies regarding the magnitude of Dynamic Earth Pressure coefficient as shown below:

The variation of the location of the vertical centroid is further highlighted by an article presented by Nicholas Sitar – Seismic Response of Slopes and Design of Retaining Structures, presented to New Zealand Geotechnical Society 2017.

Point of Load Application

Author	Point of Application
Mononobe-Okabe (1926-1929)	0.33H
Seed and Whitman (1970)	0.6H
Nandakumaran and Joshi (1973)	<0.65H
Krishna et al. (1974)	~0.5H
Sherif et al. (1982)	~0.42H
Prakash and Brasavanna (1969)	varies with acceleration
Ichihara and Matsuzawa (1973)	varies with acceleration
Ortiz et al. (1983)	varies, but higher than H/3
Woodward and Griffiths (1992)	varies with acceleration
Steedman and Zeng (1990)	varies, but higher than H/3
Mylonakis et al. (2007)	0.33H

N. Sitar, UC Berkeley 2017



N. Sitar, UC Berkeley 2017

Figure 8: N. Sitar UC Berkeley 2017

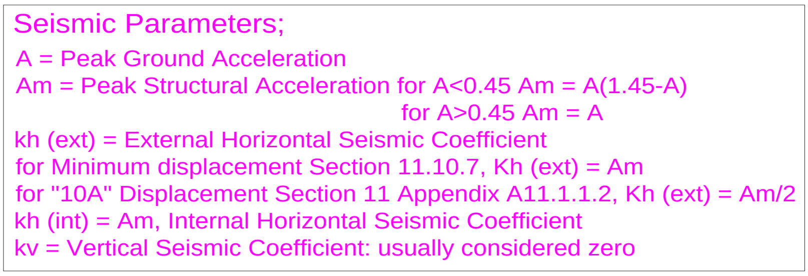
The research studies have indicated that there is a possibility that the relationship between the coefficient Pae and peak ground acceleration is linear rather that exponential as assumed in M-O theory.

The article further concluded that “M-O solution is overly conservative at high PGA>0.4-0.5 g and fails to converge for high acceleration >0.7 g with “cohesionless soil”.

While the conservative M-O solution could partly be responsible for the performance of MSE wall in seismic application, the flexibility of the MSE wall to allow mobilization of the MSE structures without damage, displacement capability could be other contributing factor.

4 MAGNITUDE OF DISPLACEMENT IN MSE

It is worth noting that the AASHTO code did commit to the magnitude of displacement in the lateral direction with the following seismic parameters and assumption:



Seismic Parameters;
A = Peak Ground Acceleration
Am = Peak Structural Acceleration for A<0.45 Am = A(1.45-A)
for A>0.45 Am = A
kh (ext) = External Horizontal Seismic Coefficient
for Minimum displacement Section 11.10.7, Kh (ext) = Am
for "10A" Displacement Section 11 Appendix A11.1.1.2, Kh (ext) = Am/2
kh (int) = Am, Internal Horizontal Seismic Coefficient
kv = Vertical Seismic Coefficient: usually considered zero

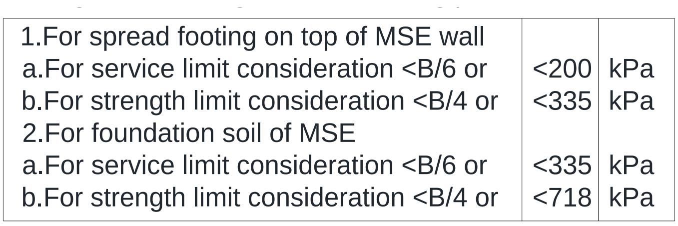
Figure 9: Horizontal displacement assumption in AASHTO 2010 for MSE

Since the components of MSE wall which are the reinforced granular fill, reinforcing elements (geosynthetic membrane) and facial elements have predictable and reliable engineering properties, it provided the opportunity for the modeling of lateral displacement due to seismic influence by finite element analysis methods (FEM) to be exploited. Such privilege however cannot be extended for the determination of the magnitude of vertical displacement (settlement and seismic thrust) on the foundation soil of MSE.

5 MSE TRUE BRIDGE ABUTMENT

As a result, while acceptable magnitude of displacement in MSE has been the salient features, the determination of the magnitude of vertical displacement be it settlement or due to seismic thrust is an issue. The modeling of FEM on the soil subgrade beneath MSE even with the constitutive model from elastic to plastic cannot be conclusive as there are too many uncertainties of variables involved.

In such a situation, the analysis is guided by limiting the maximum permissible bearing pressure and the eccentricity of the bearing pressure on the base of the abutment and on the foundation soil of the MSE:



1.For spread footing on top of MSE wall		
a.For service limit consideration <B/6 or	<200	kPa
b.For strength limit consideration <B/4 or	<335	kPa
2.For foundation soil of MSE		
a.For service limit consideration <B/6 or	<335	kPa
b.For strength limit consideration <B/4 or	<718	kPa

FHWA NHI-10-025 MSE Walls & RSS Vol II

Figure 10: Limits of bearing pressures for true bridge abutment

Furthermore since extensible reinforcements reach their peak strength at strains greater than the strain required for soil to reach its peak strength, the reinforcement for true bridge MSE abutment as such is presently confined to inextensible (steel) reinforcement.

5.1 Semi integral bridge abutment

Most of the traditional simple supported bridge seat requires large reinforced concrete approach slab to satisfy the sliding requirement when exposed to dynamic force. The introduction of MSE to the bridge approach not only relieves the bridge seat for lateral static and dynamic resisting responsibility however it allows a semi integral bridge abutment to be incorporated as well.

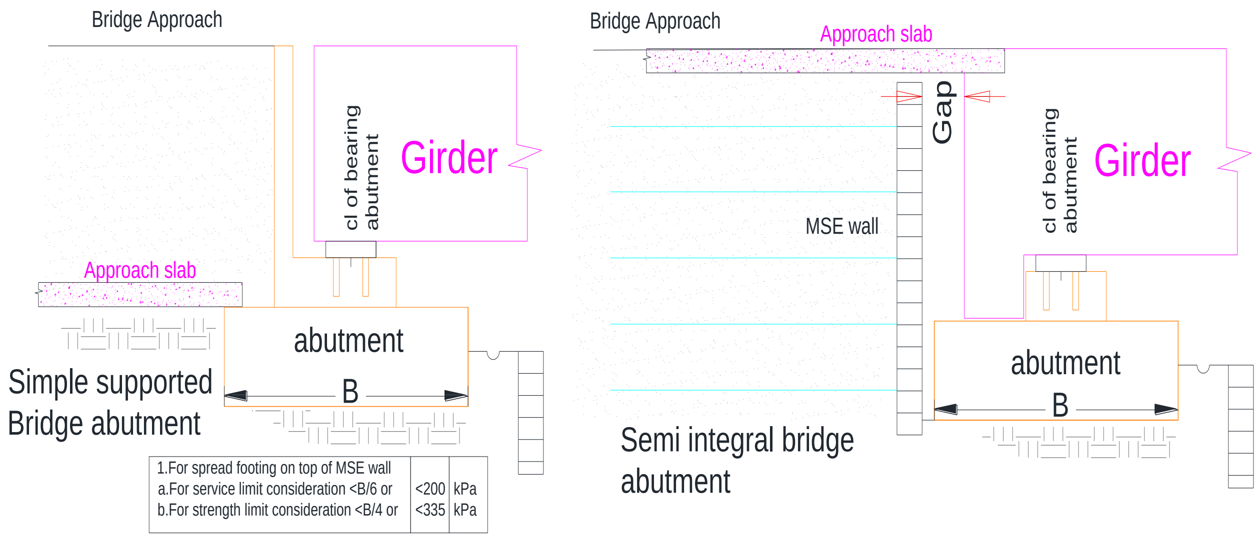


Figure 10: Simply supported and Semi integral bridge abutment

This is achieved by providing a gap detailed which is also to accommodate the daily expansion and contraction of the bridge girder.

6 GEOSYNTHETIC REINFORCED SOIL – INTEGRATED BRIDGE SYSTEM (GRS-IBS)

With the emergence of stiff geosynthetic materials, GRS-IBS is fast gaining popularity. The system is based on limiting the vertical spacing of the geosynthetic membrane to not more that 200mm, high strength low extensible geosynthetic materials and open graded reinforced fill.

7 CONCLUSION

The ability MSE structures to undergo displacement without damage suit seismic application.

To ensure that the MSE technology is accessible and appealing to practicing engineers, the design methodologies of MSE structures based on the combination of soil classical plasticity theories (Rankine) and empirical knowledge is developed and maintained.

The AASHTO codes and FHWA technical publications provide good guide for the design of MSE structures. It is as such recommended that reference be made to these documents for design of MSE structures.

Though AASHTO specified a maximum allowable differential settlement of 1%, MSE tolerates much higher differential settlement without damage.

However the application of MSE as true bridge abutment, the strict adherence to the maximum allowable of 1% differential settlement is desirable to ensure that the serviceability requirement is not compromised.

Since a high percentage of total settlement of MSE is due to the self-weight, this phenomenon is commonly exploited in the true bridge abutment structures as elevation losses can be compensated prior to the construction of the bridge abutment seat.

Nevertheless, as a guide, to exploit the self-weight of MSE for the dissipation of pore water pressure which consequently induced consolidation and settlement, the coefficient of consolidation of the foundation soil shall not be less than 10^{-3} cm^2/s.

The emergence of high strength low extensible geosynthetic has allowed accelerated bridge construction, Geosynthetic Reinforced Soil - Integrated Bridge System (GRS-IBS) to be realised.

As MSE structures are economical, simple and environmentally friendly in construction, these structures provide a good solution for retaining as well as load supporting options when site conditions suit their application.

8 ACKNOWLEDGEMENT AND REFERENCES

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