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Protective Construction with Reinforced Earth

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ABSTRACT

The objective of this research is to develop a simple analytical method that characterizes plane shock wave propagation through reinforced earth and the dynamic interaction with modular retaining wall panels. The shock wave was initiated as a velocity boundary condition. The exact solution was obtained by the Laplace transform method. A step-by-step design procedure based on the "limit state" concept is proposed. Because of the impulsive nature of ground shock, the maximum response of the wall panel and reinforced soil system depends mainly on the capacity and rate of energy absorption and dissipation of the system. Therefore, the connection between wall panels and soil reinforcement, and soil reinforcement itself should be ductile beyond the proportional limit. Furthermore, the soil reinforcement should possess a high elastic tensile modulus to minimize the wall panel displacement.

INTRODUCTION

The use of reinforced earth in the construction of retaining walls has received much attention during the past decade. A typical wall can be constructed with interlocking modular panels or blocks connected to soil reinforcement. The layers of soil reinforcement, in the form of sheets or grids placed in a backfill, usually run parallel to the direction of wave propagation. Recent field explosive tests on a reinforced earth shelter conducted in Israel(Raudanski 1990; Reid 1990,1991) have shown that such shelters can provide good protection from blast loading. Cruciform wall panels attached to horizontal metallic strips in a sandy backfill were utilized in that shelter construction.

Imposing continuity for both stress and displacement at the interface between the soil and structure, Drake and Rochefort (1987) showed that the interface stress can be expressed as

$$\sigma_i = \sigma_{ff} + \rho c_L (V_{ff} - \dot{u})$$
$$= 2\sigma_{ff} - \rho c_L \dot{u} \tag{1}$$

where ρ is the mass density and c_L the loading wave velocity of the soil, V_{ff} is the free-field particle velocity, σ_{ff} is the free-field incident stress, and $\dot{u}$ is the velocity of the structure. They also derived the equation of motion for a single-degree-of-freedom (SDOF) structural system, and presented solutions for perfectly plastic and elastoplastic structural responses.

ANALYTICAL MODEL

Figure 1 shows a one-dimensional model for the dynamic interaction between a wall panel and the reinforced earth attached to it. The presence of reinforcement in soil may significantly alter the soil's original mechanical properties. Since soil is not capable of carrying tensile stress, it is assumed in this model that any tension developed in the soil will be taken by the reinforcement and that the soil and wall panel stay bonded at the interface. The shear and bending resistance from connections between the panels has also been included in the analysis.

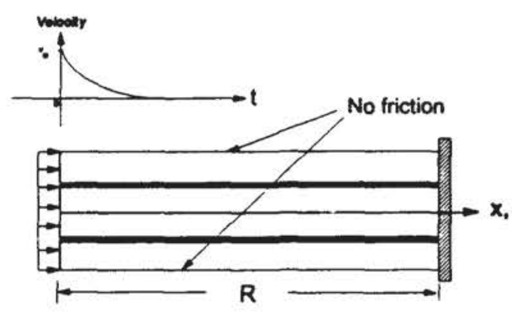

Figure 1. 1-D Model of Shock Wave Propagation Through Reinforced Earth

Governing Equation. The 1D wave equation for the particle displacement, $u(x,t)$, in a homogeneous medium is

$$\frac{\partial^2 u}{\partial t^2} = c^2 \frac{\partial^2 u}{\partial x^2} \tag{2}$$

where c is the wave propagation velocity of reinforced earth. Assuming strain compatibility, the apparent constrained modulus of the reinforced soil can be expressed in terms of the volume ratio of soil reinforcement, V_g,

$$K_x = \frac{E_s(1-\nu_s)}{(1+\nu_s)(1-2\nu_s)}(1-V_g) + \frac{E_g(1-\nu_g)}{(1+\nu_g)(1-2\nu_g)}(V_g)$$

$$-V_g(1-V_g)\frac{\left[\frac{E_s v_s}{(1+v_s)(1-2v_s)}-\frac{E_g v_g}{(1+v_g)(1-2v_g)}\right]^2}{\frac{E_s(1-v_s)}{(1+v_s)(1-2v_s)}(V_g)+\frac{E_g(1-v_g)}{(1+v_g)(1-2v_g)}(1-V_g)} \tag{3}$$

where E_s and E_g , and v_s and v_g , are, respectively, the Young's moduli and Poisson's ratios of the soil and reinforcement. The wave propagation velocity can be approximated by

$$c \approx \sqrt{\frac{K_x}{\rho_o}} \tag{4}$$

and ρ_O , is the mass density of the reinforced earth given by

$$\rho_o = \rho_s(1-V_g)+\rho_g V_g \tag{5}$$

where ρ_s and ρ_g are the mass densities of the soil and reinforcement, respectively.

Boundary Conditions. At x = 0, the shock wave front, having an initial particle velocity, v_O, arrives at time t = 0 and decays exponentially, so that

$$\frac{\partial u}{\partial t}(0,t) = v_o e^{-\alpha t} \qquad (t>0) \tag{6}$$

where α is the particle velocity attenuation rate. The shock front pressure, σ_O, is the product of the impedance of the reinforced earth, $\rho_O c$, and the initial particle velocity, v_o.

At x = R, the equation of motion of the wall panel is

$$M\frac{\partial^2 u}{\partial t^2} = -\sigma_x hb - K(u)u \tag{7}$$

where M is the mass of the wall panel, h is the panel height, b is the panel width, and $K(u)$ is the structural stiffness of the wall. Expressing soil stress in terms of the wall panel displacement, Eq.(7) becomes

$$\mu\frac{\partial^2 u}{\partial t^2}+\frac{\partial u}{\partial x}+\frac{K(u)u}{K_x hb}=0 \tag{8}$$

where

$$\mu = \frac{M}{K_x hb} \tag{9}$$

The unit resistance function, defined as the structural resistance per unit area of wall panel, can be expressed as

$$R(u) = \frac{K(u)u}{hb} \tag{10}$$

The unit resistance function $R(u)$ may be modeled as linearly elastic, elastoplastic, perfectly plastic or by some other appropriate model. However, the high strain rate of a structural system under a strong incident shock would produce perfectly plastic response, if the system were designed to be ductile. Assuming perfectly plastic wall response, then $R(u) = R_{max}H(t - t_a)$, where $H(t - t_a)$ is a Heaviside step function, $t_a = R/c$ is the arrival time of the shock wave, and the ratio of unit resistance to the constrained reinforced earth modulus becomes a constant

$$\lambda = \frac{R_{max}}{K_x} \tag{11}$$

Initial conditions. The wall panel and reinforced earth system is at rest before the shock front arrives, and thus the initial conditions are:

$$u(x,0)=0 \qquad (0\le x\le R) \tag{12}$$

$$\frac{\partial u}{\partial t}(x,0)=0 \qquad (0\le x\le R) \tag{13}$$

Solution. Eq.(2), together with boundary and initial conditions, was solved by the Laplace transform method. The solution for the particle displacement $u(x,t)$ is in the form

$$u(x,t)=\sum_1^\infty u_i \tag{14}$$

Keeping only the first five terms and using the variables,

$$t1 = t-\frac{x}{c} \tag{15}$$

$$t2 = \left(t+\frac{x}{c}\right)-2T \tag{16}$$

$$t3 = \left(t-\frac{x}{c}\right)-2T \tag{17}$$

$$t4 = \left(t+\frac{x}{c}\right)-4T \tag{18}$$

$$t5 = \left(t-\frac{x}{c}\right)-4T \tag{19}$$

$$T=\frac{R}{c} \tag{20}$$

$$m=\mu\alpha c \tag{21}$$

$$p=m+1 \tag{22}$$

$$q=m-1 \tag{23}$$

$$\gamma=\frac{1}{\mu c} \tag{24}$$

$$\delta=\mu c^2 \tag{25}$$

the terms on the right hand side of Eq.(14) take the form

$$u1 = 0 \qquad (t1<0)$$

$$= \frac{v_o}{\alpha}\left(1-e^{-\alpha\cdot t1}\right) \qquad (t1>0) \tag{26}$$

$$u2 = 0 \qquad (t2 < 0)$$

$$= -\frac{v_o}{\alpha q}\left[p\left(1-e^{-\alpha t2}\right) - 2m\left(1-e^{-\gamma t2}\right)\right] - \lambda\left[c \cdot t2 - \delta\left(1-e^{-\gamma t2}\right)\right] \qquad (t2 > 0) \qquad (27)$$

$$u3 = 0 \qquad (t3 < 0)$$

$$= \frac{v_o}{\alpha q}\left[p\left(1-e^{-\alpha t3}\right) - 2m\left(1-e^{-\gamma t3}\right)\right] + \lambda\left[c \cdot t3 - \delta\left(1-e^{-\gamma t3}\right)\right] \qquad (t3 > 0) \qquad (28)$$

$$u4 = 0 \qquad (t4 < 0)$$

$$= -\frac{v_o}{\alpha q^2}\left\{p^2\left(1-e^{-\alpha t4}\right) - 4m^2\left(1-e^{-\gamma t4}\right) + 4mq\left[1-(1+\gamma \cdot t4)e^{-\gamma t4}\right]\right\}$$
$$+\lambda\left[c \cdot t4 - \delta\left(1-e^{-\gamma t4}\right)\right] - 2\lambda\delta\left[1-(1+\gamma \cdot t4)e^{-\gamma t4}\right] \qquad (t4 > 0) \qquad (29)$$

$$u5 = 0 \qquad (t5 < 0)$$

$$= \frac{v_o}{\alpha q^2}\left\{p^2\left(1-e^{-\alpha t5}\right) - 4m^2\left(1-e^{-\gamma t5}\right) + 4mq\left[1-(1+\gamma \cdot t5)e^{-\gamma t5}\right]\right\}$$
$$-\lambda\left[c \cdot t5 - \delta\left(1-e^{-\gamma t5}\right)\right] + 2\lambda\delta\left[1-(1+\gamma \cdot t5)e^{-\gamma t5}\right] \qquad (t5 > 0) \qquad (30)$$

The expressions for the longitudinal normal stress, particle velocity, particle acceleration of the soil medium can be readily derived from Eqs. (26)-(30). Although higher order terms could be added to the solution, the transient response of the reinforced soil system due to shock loading will have been damped out before they become effective. The 1-D model accounts for superposition of incident and reflected waves propagating between the explosion point and the interface, and accommodates Eq. (1) as a special case. The 1-D model gives the same interface stress as that given by Eq. (1) when the soil medium is semi-infinite. The 1-D model is used as the basis to develop a procedure for the design of a modular wall panel and soil reinforcement connection system.

DESIGN CONSIDERATIONS

When the deceleration of the panel by the structural resistance from panel connection and soil reinforcement is less than the deceleration of the incident shock, the interface stress becomes tensile and the wall panel tends to pull the soil reinforcement out from the soil. The soil reinforcement will have to carry the tension developed at the interface. The equation of motion of the wall panel, Eq.(7), becomes

$$M\frac{\partial^2 u}{\partial t^2} + R_{max}bh + T_g = 0 \qquad (31)$$

where T_g is the tension in the soil reinforcement at the interface. If this interface tension is greater than the "pull-out resistance" of the soil reinforcement, slippage will occur between soil and soil reinforcement and the wall panel will become separated from the soil. The confined stiffness or "pull-out resistance" of the soil reinforcement should be used in the design calculations. Figure 2 shows different characteristics of the confined versus unconfined stiffnesses of CONWED Stratagrid™ 9027(Farrag et al. 1991).

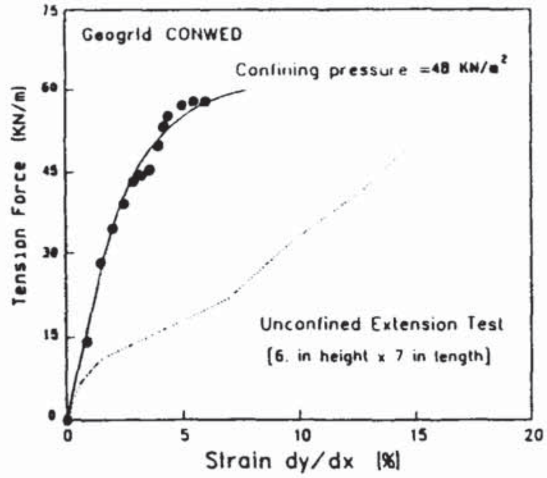

Figure 2. Confined vs. Unconfined Stiffness of a Geogrid

The higher confined stiffness can be attributed to the composite action of the reinforcement with surrounding soil. If the wall panel stays in contact with soil, the connection system is termed "compression-controlled". If the wall panel separates from the soil, the connection system is termed "tension-controlled". Figure 3 shows the relationship between the ratio γ / α and the ratio σ_o / R_{max}, which can be used to determine whether separation will occur.

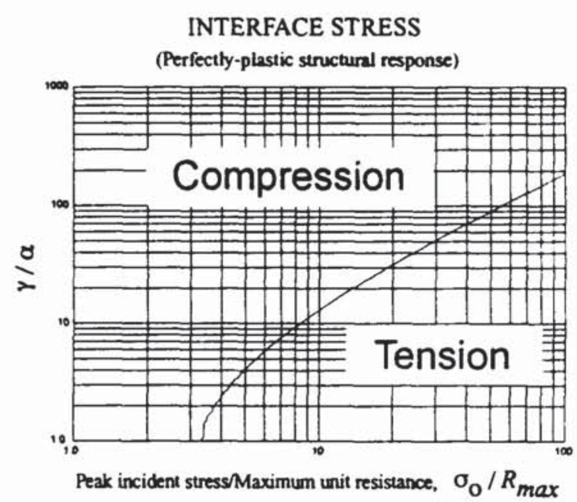

Figure 3. Prediction of Tensile or Compressive Interface Stress

The free-field soil displacement under the shock wave prescribed by Eq.(6) is

$$u_{ff}(t) = \frac{v_o}{\alpha}(1-e^{-\alpha t}) \qquad (32)$$

The maximum wall panel displacement, u_{max}, for a compression-controlled system is always less than twice the peak free-field soil displacement. This corresponds to the limiting case of a free soil boundary where $\lambda \to 0$, $\mu \to 0$ and $\gamma \to \infty$. However, a large wall panel displacement may occur for a tension-controlled system. When the wall panel becomes separated from the soil, slippage between soil and soil reinforcement will have occurred. If the soil reinforcement tension at the interface is assumed to be a constant, being equal to the smaller of the soil reinforcement yielding force or the dynamic frictional resistance between soil and soil reinforcement, the maximum panel displacement can be determined by solving Eq.(31) numerically. In this case, the unit resistance R_{max} is the combined resistance of the panel connection and soil reinforcement. Figure 4 shows a normalized displacement envelope in terms of σ_o / R_{max} for both compression- and tension-controlled systems.

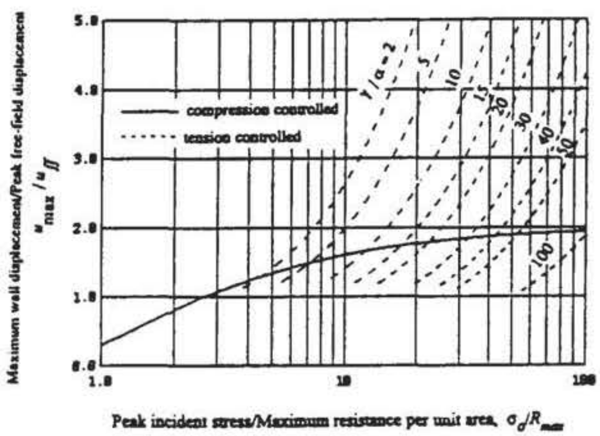

Figure 4 Ratio of Maximum Wall Displacement to Maximum Free-field Soil Displacement

EXAMPLE

A numerical example is given herein to illustrate how the simple 1-D model can be used to determine the normal stress acting on the interface between the reinforced earth and the wall panel and the kinematic response of the wall panel. Furthermore, the effects of wall panel separating from the soil on these response parameters are also presented. The physical parameters used in this example are given in Table 1. Figures 3 and 4 may be used to determine the maximum wall panel displacement rapidly. Using Eq.(3) and the values given in Table 1, the apparent constrained modulus of the reinforced earth, K_x, is computed to be 59,427 psi (410 MPa). Since the soil is very lightly reinforced, the mass density of the soil is not significantly affected by the presence of the reinforcement. The seismic velocity of the reinforced earth, c, is computed to be 1583 fps (482 m/s). Combining Eqs.(9) and (24) yields

$$\gamma = \frac{K_x hb}{Mc} \approx 2000 \ Sec^{-1} \tag{33}$$

and $\gamma / \alpha = 25$. The peak normal stress is computed from the free-field particle velocity,

$$\sigma_o = \rho_o c \, v_o = 230 \ psi \tag{34}$$

and $\sigma_o / R_{max} = 46$ From Figure 3, the connection system falls in the "tension-controlled" region. This means that, during the loading phase, the wall panel is likely to separate from the soil. Using Figure 4, the ratio of maximum wall panel displacement to the peak free-field displacement is found to be approximately 2.5. The peak free-field soil displacement is determined using Eq.(32),

$$u_{ff} = \frac{v_o}{\alpha} = 0.92 \ in. \quad (23 \ mm) \tag{35}$$

and the maximum wall panel displacement, u_{max}, is $0.92 \times 2.5 = 2.3 \ in. \quad (58 \ mm)$

Table 1. 1-D Model Parameters for the Example

Parameter (1)	Value (2)
Young's Modulus of Soil, E_s	255 MPa (37,000 psi)
Poisson's Ratio of Soil, v_s	0.35
Dry Unit Weight of Soil, ρ_s	1765 kg/m^3 (110 pcf)
Young's Modulus of Reinforcement, E_g	1.08 GPa (157,000 psi)
Unit Weight of Reinforcement, ρ_g	963 kg/m^3 (60 pcf)
Poisson's Ratio of Reinforcement, v_g	0.40
Volume Ratio of Reinforcement, V_g	0.024%
Free-field Soil Particle Velocity, v_o	1.8 m/s (6 fps)
Exponential Decay Rate, α	80 Sec^{-1}
Length of Reinforced Earth, R	6.1 m (20 ft)
Seismic Velocity of Soil, c	488 m/s (1600 fps)
Unit Structural Resistance, R_{max}	35 kPa (5 psi)
Height of Wall Panel, h	154 cm (5 ft)
Width of Wall Panel, b	180 cm (6 ft)
Weight of Wall Panel, M	1180 kg (2600 lb)

Figure 5 shows that the interface stress is the superposition of the incident stress and the structural unit resistance.

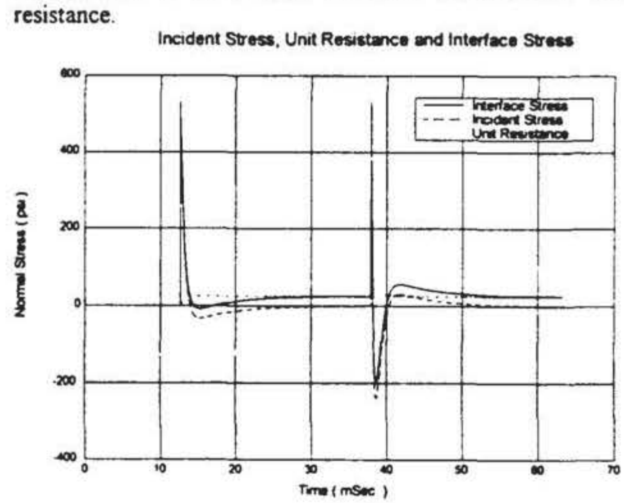

Figure 5 Composition of Interface Stress

In this example, the interface stress becomes tensile at time t = 14.36 mSec when the wall panel starts separating from the soil. Eq.(31) becomes effective from that time instant and is solved numerically to determine the maximum wall panel displacement. Figure 6 shows that wall panel displacement would be significantly undcrestimated if Eqs.(26)-(30) were used for a "tension-controlled" connection system.

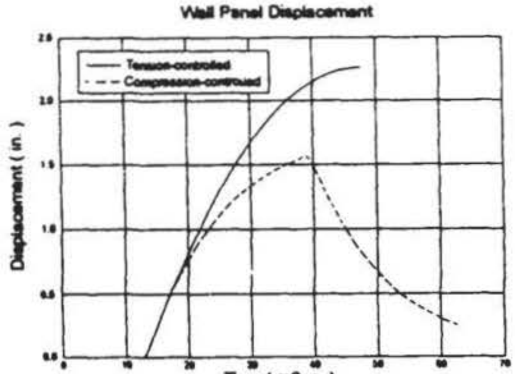

Figure 6. Effect of Panel Separation from Soil on Wall Panel Displacement

LIMIT STATE DESIGN PROCEDURE

Using the above information, a step-by-step design methodology is illustrated herein:

Step 1. Determine the volume ratio of reinforcement in the reinforced soil, V_g.

Step 2. Determine the peak value σ_o and the decay rate α of the free-field normal stress in the reinforced soil due to a given explosion.

Step 3. Conduct limit analyses of the maximum pull-out resistance of reinforced soil.

The resistance of the reinforced soil may become ineffective if sufficient embedment length is not provided to develop the required tensile force in the reinforcement. Due to the high strain rate from a ground shock, an ideal soil reinforcement should posses high tensile modulus, high tensile and impact strength, but most importantly, high ductility. Requirements for minimum embedment length and maximum vertical spacing of reinforcement layers have been developed based on slope stability by Christopher et al. (1990) and Jewell (1990). The maximum pull-out resistance of the reinforced soil is dictated by the following three modes of failure:

- soil shear failure in a zone away from the reinforcement;
- tensile rupture in the reinforcement; and
- bond failure between soil and reinforcement.

The smallest value of these resistances is the maximum pull-out resistance of the reinforced soil.

Step 4. Conduct limit analyses of the maximum resistance of wall panel connection.

Step 5. Determine the maximum wall panel displacement due to the given ground shock

For perfectly plastic structural response, the maximum resistance per unit area of wall panel (or unit resistance), R_{max}, is the sum of the maximum unit resistance of the reinforced soil and that of the wall panel connection. Figures 3 and 4 can then be used to determine the maximum wall panel displacement.

Step 6. Design against breaching of wall panels

The 1989 version Air Force Protective Design Manual provides guidelines for determining structural element thickness and minimum standoff distance to prevent localized breaching. McVay(1988) reported that breaching is likely to occur when the scaled range, $R/W^{1/3}$, is less than 1.3 ft/lb$^{1/3}$, where R is the standoff in feet and W is the net explosive weight in pounds of TNT. In general, the concrete wall panels are reinforced with welded wire fabric(WWF) made of A82 steel with a minimum yield strength of 64000 *psi*. For close-in and contact explosions, fibrous concrete may be used as an alternative.

CONCLUSION

The 1-D mathematical model proposed in this study provides a simple method for predicting the dynamic interaction between reinforced earth and wall panels under ground shock loading. The model accounts for superposition of incident and reflected waves propagating between the velocity boundary and the wall panel. The model treats reinforced earth as a linearly elastic and homogeneous medium, and as such cannot model the hysteretic compaction or other plastic behavior of soil under stress wave propagation.

This method of analysis can be applied to earthquake engineering, where the ground motion may be treated as a series of shock waves arriving at the wall at different instants of time. It is anticipated that wall panel will not separate from the soil at low stress level, and therefore, the responses to the different shocks can be superimposed to find the wall response under an earthquake motion.

ACKNOWLEDGMENT

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