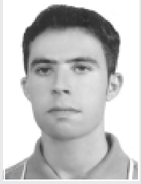
Proceedings of the Institution of Civil Engineers
Ground Improvement 163
May 2010 Issue GI2
Pages 121–132
doi: 10.1680/grim.2010.163.2.121

Paper 700044
Received 02/11/2007
Accepted: 18/08/2009

Keywords: foundations/ mathematical modelling



Mehdi Heidari
Graduate student, Shiraz University, Shiraz, Iran

M. Hesham El Naggar
Professor, Department of Civil and Environmental Engineering, University of Western Ontario, London, Canada

ice
Institution of Civil Engineers

Using reinforced soil systems in hammer foundations

M. Heidari and M. H. El Naggar

Forging hammers produce powerful short-period impact loads. A mounting system or a properly designed foundation could be used to transmit the shock to the soil and lessen its effects on the surroundings. The supporting foundations must be designed to reduce the vibration amplitude and the forces transmitted to the soil medium in order to meet serviceability and stability requirements. Soil reinforcement may be used to improve the performance of the foundations supporting shock-producing equipment. This paper investigates the effect of soil reinforcement on the performance characteristics for different configurations of shock-absorbing foundations. The results demonstrated the efficiency of soil reinforcement in improving the performance of foundations subjected to impact load. A parametric study was conducted and a set of charts was established as practical guidance for the design of soil reinforcement schemes.

1. INTRODUCTION

The act of hammering is a basic industrial process encountered in such diverse activities as the breaking up of scrap and the forging of metal. Hammers, presses and other types of shock-producing machines generate powerful dynamic effects that are quite short in duration and can be characterised as pulses. Only a part of the shock energy is utilised in the intended machine function and the rest is dissipated in the foundation causing intense vibration. Shock-absorbing foundations such as mounting systems are now used to support hammers and to reduce the transmission of the impact force to the soil.

The shock is transmitted through the mounting system and foundation to the soil and surroundings. Heavy shocks imparted to the foundation can cause alignment problems (i.e. reduce operating life), neighbour complaints and prohibit proper operation of adjacent equipment. Therefore, the main objectives of the design of a foundation supporting shock-producing equipment are to reduce the vibration amplitudes and the forces transmitted to the soil and/or to minimise any disturbance to the neighbourhood and surroundings.

Current practice is to install die forgers and hammers on layered elastomeric isolators or viscous spring isolators. The foundation system for forging hammers using elastomeric and spring isolators is shown in Figure 1.

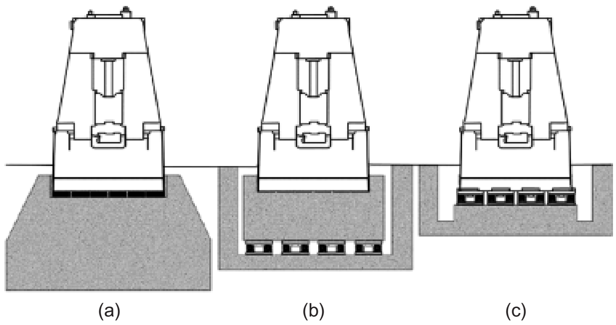


Figure 1. Isolated foundation types for forging hammers: (a) layered elastomer; (b) isolator supported inertia block; (c) direct isolator support

The impact force created by each blow of the forging hammer is absorbed by the vertical motion of the machine through viscous damping. The layered elastomeric foundation systems, as shown in Figure 1(a), and the viscous spring isolator mounting systems, as shown in Figures 1(b) and (c) are designed to allow the motion of the machine to decay below a certain level before the next impact.

To ensure satisfactory performance of the machinery, the mounting system and/or the foundation should be designed such that the vibration amplitudes do not exceed the values given in Table 1 (Novak, 1983).

A number of dynamic models have been developed to analyse the response to pulse loading of one-mass and two-mass foundations with springs and dampers. Using these models, the influence of various parameters of the isolator mounting system was studied (e.g. Chehab and El Naggar, 2003, 2004;

Hammer weight: t	Anvil		Foundation block	
	mm	in.	mm	in.
< 1	1	0·04	1·2	0·05
2	2	0·08	1·2	0·05
> 3	4	0·16	1·2	0·05

Table 1. Maximum allowable amplitude for hammer foundations

El-Hifnawy and Novak, 1984; Heydari *et al*., 2008; Novak and El-Hifnawy, 1983; Wang and Dong, 2006). Novak and El-Hifnawy (1983) examined two methods based on energy consideration and a complex eigenvalue approach to incorporate damping in the analysis of the response of one-mass and two-mass hammer foundation systems. The effect of the anvil pad flexibility on the foundation response was examined. They studied the undamped and damped responses of two-mass foundation systems to pulse loading. Chehab and El Naggar (2003) investigated the efficiency of impact isolation for different anvil and machine configurations considering two-mass foundation systems. A parametric study was conducted to reveal the influences of stiffness and damping of a mounting system on the dynamic behaviour of hammer foundations. The effects of the pulse shape and pulse duration on the dynamic response of the one-mass hammer foundation system have also been investigated (Chehab and El Naggar, 2004). A new method for performing design optimisation of a viscous spring isolator mounting system for a forging hammer was introduced (Wang and Dong, 2006). Heydari *et al*. (2008) investigated the extent of efficiency of soil reinforcement on the dynamic responses of a machine foundation under vertical vibration for different vibration isolation systems.

Mounting systems may not always achieve satisfactory performance. The optimum use of mounting systems depends on the intended purpose, the mass arrangement and the dynamic characteristics of foundation. Thus, the dynamic soil properties can be altered to improve the performance of foundations supporting shock-producing equipment.

Founding a footing on a suitably reinforced soil medium can lead to considerable improvement of its bearing capacity and significant reduction of its settlement (Shin *et al*., 2002). Jones (1985) proposed a model for the confinement effect of reinforcement and used it for the analysis of a reinforced foundation bed. The confinement effect is quantified in terms of the average increase in confining pressure due to the reinforcement, which is used to evaluate the modified shear stiffness of the granular soil surrounding the reinforcement (Ghosh and Madhav, 1994). Furthermore, soil reinforcement improves the dynamic properties of the soil (Montanelli and Recalcati, 2003; Shuwang *et al*., 2004), increases the stiffness of piled foundations (El Naggar and Abdel-Meguid, 1997) and has been found to improve the response of footings to harmonic loading (El Naggar and Wei, 1997). Al-Dobaissi (1990) examined the resistance of reinforced soil to impact loading of different magnitudes in a laboratory testing programme. The test results revealed the superior performance of reinforced soil.

Reasonable determination of shear modulus of the supporting medium is necessary for the design of satisfactory and reliable foundations subjected to low strain vibrations (e.g. machinery, traffic, paper mills and compressor stations). The elastic modulus of soil (and its shear modulus) can be increased by reinforcement and the settlement of the reinforced soil can be noticeably decreased (Al-Dobaissi, 1990). Therefore, a reasonable estimate of the response of a reinforced soil foundation can be made by determining the increased value of homogenised shear modulus of the reinforced soil. It is important, however, to realise that the magnitude of shear modulus of reinforced soil is a function of several parameters.

The non-linear elastic model for fibre-reinforced soils under cyclic loading at small strain were introduced and the effects of geofibre, confining pressure and loading repetition on the elastic shear modulus of reinforced soil were studied and analysed (Li and Ding, 2002). Das and Maji (1994) and Das *et al*. (1998) conducted laboratory model tests to study the settlement of a square foundation supported by geogrid-reinforced sand and subjected to transient load. They found that the geogrid reinforcement reduced the settlement of the foundation. The ultimate settlements due to the transient loading for both reinforced and unreinforced soils were calculated and their ratio was defined as the settlement reduction factor, R; and found to be a function of reinforcement depth. More recently, the effect of the location and the number of reinforcement layers on low strain stiffness and bearing capacity of shallow foundations was studied. The results of both laboratory and numerical tests indicate that the soil stiffness at low strain levels and bearing capacity can be greatly improved with reinforcement layer(s) placed underneath the foundation at the critical and most effective location(s). It was concluded that soil reinforcement can also be used to reduce low strain vibrations of foundations (Chung and Cascante, 2007).

Soil reinforcement is therefore considered a potentially advantageous technique to enhance the performance of hammer foundation systems under impact loads. In the present study, the effect of soil reinforcement on the foundation response was examined for different configurations of shock-absorbing foundation. The results were used to provide some guidance for the design of an appropriate reinforced foundation with shock-absorbing system for a given application.

2. REINFORCED SOIL FOUNDATION

The use of geosynthetics to improve the bearing capacity and settlement performance of shallow foundations has proved to be a cost-effective foundation system. A reinforced soil foundation (RSF) consists of one or more layers of a geosynthetic reinforcement and controlled fill placed below a conventional spread footing to create a composite material with improved performance characteristics. A composite reinforced soil foundation (CRSF) is an RSF that also includes a geosynthetic fabric separating native soil from the fill used to construct the RSF. Reinforced soil foundations may be used to construct shallow foundations on loose granular soils, soft fine-grained soils, or soft organic soils. Most RSFs are constructed with the reinforcement placed horizontally; however, there are cases in which vertical reinforcement may be used. The reinforcement may consist of geogrids, geofabrics, geocells or other geosynthetics.

Considerable advances have been made into the understanding of the behaviour of reinforced soil foundations and on the applications and limitations of using geosynthetics to improve the performance of shallow foundations. Detailed investigations have been performed using small- and large-scale model footings to evaluate the performance of RSFs and to develop rational methods for design.

The fill placed between layers of reinforcement is usually a clean coarse road-base material that is compacted to a minimum relative density of about 75%, but may also consist of compacted sand. A number of factors may influence the performance of an RSF, including: (*a*) type of reinforcement; (*b*) number of reinforcing layers in the zone of influence, N; (*c*) depth below the footing to the first layer of reinforcement, u/B; (*d*) spacing between reinforcing layers, h/B; (*e*) width of reinforcement layers, b/B; (*f*) total depth of reinforcement, d/B; (*g*) type of imported loads; and (*h*) type and placement of the fill. Some of these parameters for geogrid-reinforced sand are shown in Figure 2.

Several model studies have been conducted on shallow foundations to determine the optimum values of u/B, d/B, b/B and h/B in order to obtain the maximum benefit of soil reinforcement (e.g. Guido *et al.*, 1987; Omar *et al.*, 1993). The model test results have shown that, for given values of u/B, h/B and b/B, the magnitude of d/B for consideration of the stress influence as related to bearing capacity and settlement is about 1·5 for square foundations, and increases to about 2 for strip foundations. Similarly, for given values of u/B, h/B and d/B, the magnitudes of the ultimate bearing capacity and settlement of foundation improve with b/B to an approximate maximum that was reported to be 4 to 4·5 and remains constant thereafter. Uchimura *et al.* (2006) evaluated the stiffness and residual deformation of a geosynthetic-reinforced soil structure subjected to sustained concentrated vertical loading and a long-term vertical cyclic loading history. They found that the effects of reinforcement stiffness were utterly insignificant on the behaviour during unloading and reloading for the range of the examined limit of reinforcement stiffness.

3. DYNAMICS OF HAMMER FOUNDATION SYSTEM

The dynamic performance of isolated foundations can be modelled mathematically using either analytical or numerical approaches. The key factors that determine the response of isolated foundation systems are: the dynamic characteristics of isolator; duration, magnitude and shape of pulse loading; size and mass of foundation; and the soil conditions. These factors are reflected in the system stiffness and damping values.

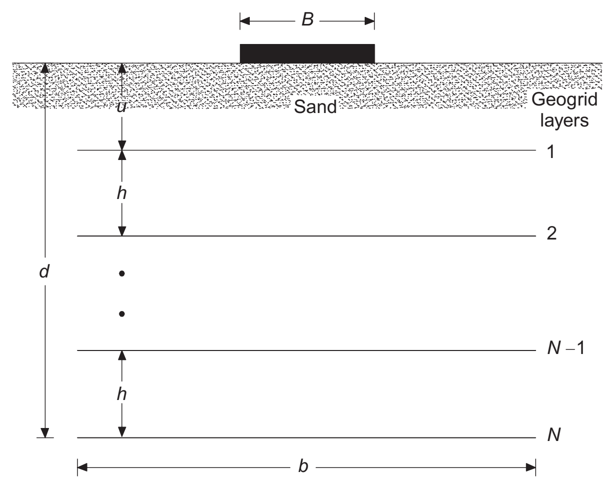


Figure 2. Geometric parameters of one type of reinforced soil foundation

3.1. Mathematical model

Several components of shock-absorbing foundation such as hammer, anvil, machine frame, viscous spring isolator, foundation block and the underneath soil should be considered in the model. With centric blows and a symmetrical arrangement, the mathematical model of hammer systems similar to those shown in Figure 1 has two degrees of freedom as shown in Figure 3. The governing equations of the system are

$$\begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix} \begin{Bmatrix} \ddot{x}_1 \\ \ddot{x}_2 \end{Bmatrix} + \begin{bmatrix} c_1 & -c_1 \\ -c_1 & c_1 + c_2 \end{bmatrix} \begin{Bmatrix} \dot{x}_1 \\ \dot{x}_2 \end{Bmatrix} + \begin{bmatrix} k_1 & -k_1 \\ -k_1 & k_1 + k_2 \end{bmatrix} \begin{Bmatrix} x_1 \\ x_2 \end{Bmatrix} = \begin{Bmatrix} f_{(t)} \\ 0 \end{Bmatrix} \tag{1}$$

where m_1 is the mass of the machine (Figure 1(a) and (c)) or the mass of the machine plus foundation block (Figure 1(b)). The mass m_2 is the mass of the foundation block and all the parts attached to it (e.g. Figure 1(a)) or the mass of the trough (Figure 1(b) and (c)). The stiffness and damping constants k_1 and c_1 represent the stiffness and damping of the spring and dashpot of the shock-absorbing system; similarly, k_2 and c_2 are the stiffness and damping coefficients representing the foundation. The anvil (or machine) and foundation block (or trough) responses are $x_1(t)$ and $x_2(t)$. Finally, $f(t)$ is the impact force acting on the anvil.

3.2. Stiffness and damping constants of the system

The prediction of the response of the hammer foundation requires the description of the stiffness and damping of the foundation and the isolators.

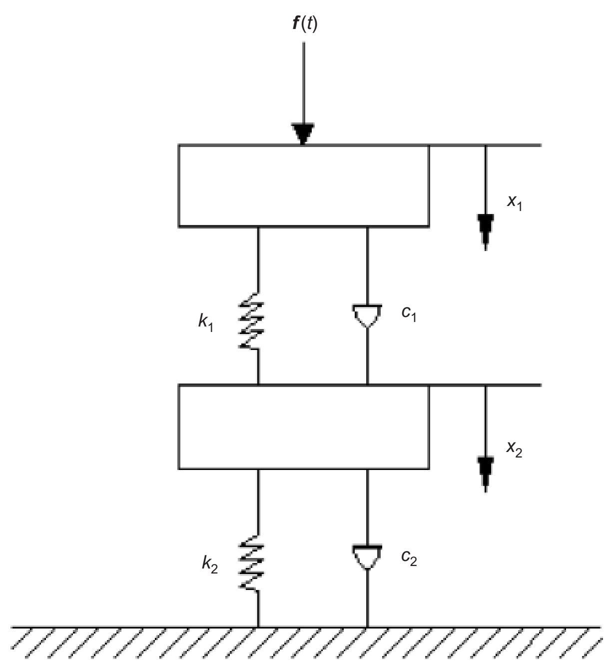


Figure 3. Two-mass hammer foundation and its vibration model

3.2.1. Isolator stiffness and damping. When the foundation block or the anvil rests on a pad of viscoelastic material, the dynamic characteristics of the pad can be computed and expressed as

$$K_1 = \frac{E_p A_p}{d}, \quad C_1 = \frac{\tan \delta_p k_1}{\omega_0} \tag{2}$$

where E_p is the Young's modulus of the pad material; A_p and d are the cross-sectional area and the thickness of the pad, respectively; $\tan \delta_p$ is the pad material damping ratio; and ω_0 is the anvil natural frequency expressed in radians per second.

Mounting systems offer the greatest degree of vibration and shock isolation because they are relatively soft when compared with elastomeric isolation systems. The stiffness and damping constants of viscous spring isolators are supplied by their manufacturers.

3.2.2. Foundation stiffness and damping. In a reinforced foundation bed under dynamic loading, it is imperative that the effect of the presence of geosynthetics is properly included. Kavazanjian and Matasovic (1995) showed that ignoring the effect of geosynthetics leads to overestimating the accelerations. The slip deformations that may occur along geosynthetic interfaces experiencing dynamic loading can limit the acceleration transmitted through the interface to the soil (Yegian and Harb, 1995). Yegian *et al.* (1998) modelled the geosynthetic interfaces using an equivalent soil layer and represented its stiffness and damping using an equivalent spring and dashpot system to simulate the behaviour of the geosynthetic–soil interface under dynamic loading. The dynamic parameters of the equivalent layer (i.e. equivalent mass, stiffness and damping) would depend on the soil layer thickness, soil modulus, and material damping ratio. Shuwang *et al.* (2004) employed this approach to analyse the soil–geogrid interaction under automobile loading.

The foundation block for a hammer can be situated on top and directly supported by the soil. However, it is usually embedded to increase the damping provided through the soil layers adjacent to the foundation sides. For embedded foundations in a deep homogeneous stratum (halfspace), the stiffness and damping coefficients can be calculated by (Novak, 1974; Novak and Beredugo, 1972)

$$K_2 = G r_0 \left(C_{v_1} + \frac{G_s}{G} \frac{\ell}{r_0} S_{r_1} \right) \tag{3a}$$

$$C_2 = r_0^2 \sqrt{\rho G} \left(\overline{C}_{v1} + \overline{S}_{v2} \frac{\ell}{r_0} \sqrt{\frac{\rho_s}{\rho} \frac{G_s}{G}} \right) \tag{3b}$$

where G is the soil shear modulus, r_0 is the base radius for circular bases or the equivalent radius ($r_0 = \sqrt{A/\pi}$ for non-circular bases), ρ is the soil density, l is the embedment depth, and G_s and ρ_s are the shear modulus and density of the side layers (backfill) respectively.

The dimensionless stiffness and damping parameters C_{v1} and C_{v2} depend on the dimensionless frequency, where ω_0 is the system natural frequency and V_s is the soil shear wave velocity. S_{v1} and S_{v2} are the dimensionless stiffness and damping parameters for the side layer. The values of C_{v1}, C_{v2}, S_{v1} and S_{v2} for most foundation types are given in Table 2 (Das, 1992).

The first set of parameters in Table 2 is used in the analysis reported herein. The effect of reinforcement is included in the analysis by substituting into Equation 3 the homogenised shear modulus, G_r, and homogenised density, ρ_r, of the reinforced soil. The geogrid could have a density between 1200 and 1700 kg/m^3, which is close to the common values of soil density. Therefore, the equivalent density of reinforced backfill may change only slightly, such that it can be assumed to remain constant.

3.2.3. Determination of homogenised shear modulus. Soil reinforcement may increase the magnitude of homogenised shear modulus of reinforced soil. Studies on the mechanical behaviour of reinforced soil are relatively recent. Li and Ding (2002) conducted cyclic shear tests using a conventional dynamic triaxial apparatus to investigate the non-linear behaviour of geofibre-reinforced soil at small strain. They reported that the elastic shear modulus of geofibre-reinforced soil increases with the increase of fibre content and confining pressure, and decreases with the increase of loading repetition.

The magnitude of the shear modulus of soil, reinforced with geosynthetics, is a function of several parameters as discussed in Section 2. Based on previous studies, typical design parameters for the use of reinforcement layers are $u/B = h/B = 0{\cdot}15–0{\cdot}03$, $b/B = 2{\cdot}0–3{\cdot}0$, $N = 2–4$, and maximum recommended values are $u/B = h/B = 0{\cdot}5$, $b/B = 5$ and $N = 5$ (Chung and Cascante, 2007). Chung and Cascante (2007) summarised the results of past laboratory studies using soil reinforcement on shallow foundations and performed a number of meticulous laboratory tests to investigate the effect of reinforcement on the low strain stiffness of shallow foundations. The slope of the linear part of the load–displacement curve for a reinforced foundation was used to determine the low-strain stiffness and normalised with the corresponding slope in the unreinforced case. A dimensionless factor, called stiffness improvement factor (*SIF*), was defined to evaluate the improvement in the low-strain stiffness of the system. The effects of the number of reinforcement layers (N) and reinforcement location were studied by conducting a set of load tests on shallow square foundations on dry sand. Their results identify a critical zone between 0·3 and 0·5B for

Soil	Halfspace (deep stratum)		Side layer	
Poisson's ratio, μ_s	C_{v1}	C_{v2}	S_{v1}	S_{v2}
0·0	3·9	3·5	2·7	6·7
0·25	5·2	5·0	2·7	6·7
0·5	7·5	6·8	2·7	6·7

Table 2. Practical stiffness and damping parameters for hammer foundation

maximising the benefits of soil reinforcement. The use of multiple layers of reinforcement is effective only if the first reinforcement layer is placed at the critical depth and the spacing between reinforcement layers is smaller than $h = 0{\cdot}3B$. When the reinforcement is located at a depth of $1{\cdot}0B$ below the foundation, the effect of reinforcement tends to disappear. It was revealed that the effect of location and number of reinforcement layers is interrelated. A summary of their laboratory results is presented in Table 3. These experimental results are valid for coarse or cohesionless material with a linear increase in strength as a function of depth.

Using the results obtained from Table 3, a non-linear regression is carried out to derive an expression that relates the *SIF* only to the number and location of reinforcement layers

$$SIF = \frac{N^{0{\cdot}74}}{\left[1{\cdot}13\left(\frac{u}{B}\right)^{0{\cdot}84} + 0{\cdot}31\left(\frac{h}{B}\right)^{0{\cdot}48}\right]} \tag{4}$$

In the above equation, the effects of width and tensile stiffness of the reinforcement layers are not considered. It should be noted that *SIF* increases with an increase in the tensile stiffness of the reinforcement (Chung and Cascante, 2007). Further study is needed to determine the optimum values of u/B, b/B, h/B and tensile stiffness of the reinforcement in order to derive the maximum benefit for improving the shear modulus of the reinforced soil below the foundation.

Cyclic plate load tests can also be carried out to evaluate the variation of the shear modulus of the reinforced soil. For the same maximum depth of reinforcement, the shear modulus increases with the number of layers in place. Cyclic plate load tests performed in the field can be used to determine the modulus of elasticity of soil (E_s) supporting the foundation. The soil shear modulus G_s can then be calculated from

$$G_s = \frac{E_s}{2(1+\mu_s)} \tag{5}$$

where μ_s is the Poisson's ratio of the soil.

The field cyclic plate load tests are conducted by applying step loads to a test plate as a sequence of unloading and reloading. In this manner, the elastic rebound of the soil, S_e, at any stress level $\Delta\sigma$ can be determined as shown in Figure 4. The variation of $\Delta\sigma$ with S_e can be used to calculate the effect of elastic uniform compression, C_z, of the soil as (Figure 5)

$$C_z = \frac{\Delta\sigma}{S_e} \tag{6}$$

The magnitude of C_z is proportional to the square root of the area of the test plate. The theoretical relationship for C_z is of the form (Prakash, 1981)

$$C_z = 1{\cdot}13\frac{E_s}{1-\mu_s^2}\frac{1}{\sqrt{A}} \tag{7}$$

Combining Equations 5 and 7 yields

$$G_s = \frac{C_z(1-\mu_s^2)\sqrt{A}}{2{\cdot}26} \tag{8}$$

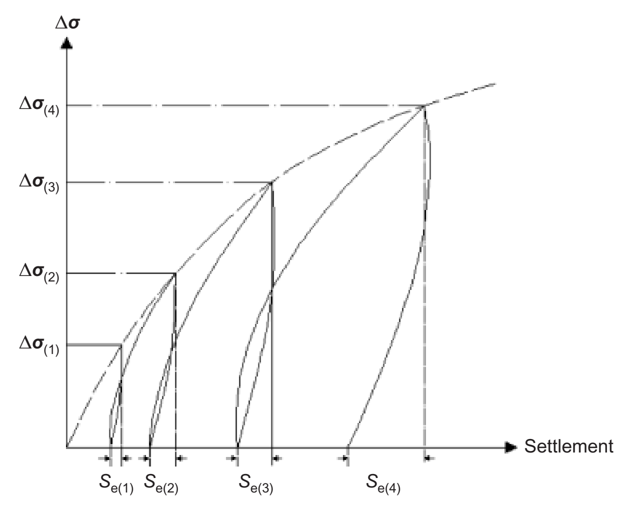


Figure 4. Plot of $\Delta\sigma$ with settlement for a cyclic plate load test

Test No.	Reinforcement	Settlement: mm	Vertical stress	Stiffness: kN/mm	*SIF*
A-1	Unreinforced	5	55·4	0·157	1·0
A-2	0·3*B*	5	113·5	0·369	2·4
A-3	0·5*B*	5	98·3	0·262	1·7
A-4	0·75*B*	5	60·9	0·160	1·0
A-5	1·0*B*	5	59·5	0·156	1·0
A-6	0·3–0·5*B*	5	171·6	0·492	3·1
A-7	0·5–0·75*B*	5	103·8	0·326	2·1
A-8	0·5–1·0*B*	5	101·0	0·311	2·0
A-9	0·75–1·0*B*	5	60·9	0·220	1·4
A-10	0·3–0·5–0·75*B*	5	227·6	0·607	3·9
A-11	0·5–0·75–1·0*B*	3·5	108·0	0·467	3·0

Table 3. Summary of laboratory results (square foundation on dry sand)

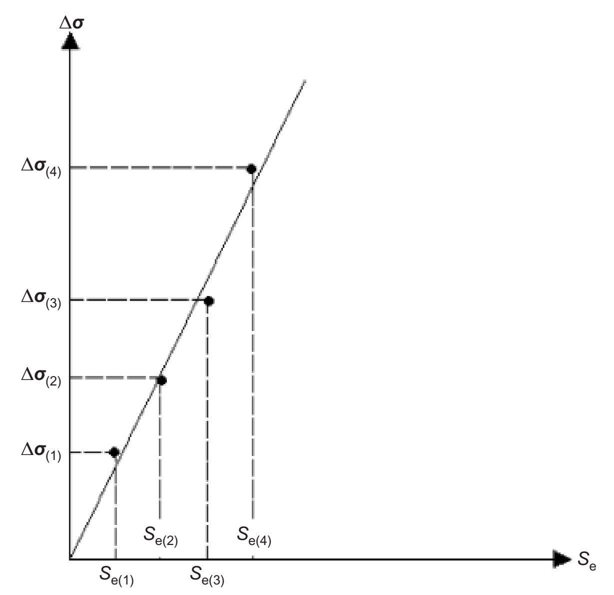


Figure 5. Plot of $\Delta\sigma$ with S_e for a cyclic plate load test

In most cases, the magnitude of G_s, as determined by Equation 8 will be for a strain level about 10^{-4} to 10^{-3} (Prakash, 1981). This strain level is representative of the elastic settlement of shallow foundations. The dynamic response of shallow foundations supported by granular material is not sensitive for Poisson's ratio, μ_s (i.e. where μ_s is less than 0·4), thus its value can be reasonably assumed.

3.3. Impact load

During normal operations, the hammer usually impacts on the anvil for a few milliseconds resulting in a transient load, $f(t)$. As the exciting force due to a hammer operation is stochastic in nature, the load–time history can not be defined exactly. However, it is commonly modelled as a half-sine pulse, triangular pulse or rectangular pulse (used in the current study). The duration and maximum size of impact are specified as the design load for the foundation system. The hammer blow can be considered in the response analysis using an initial velocity imparted to the anvil or as a pulse depending on the duration of the collision between the head and the anvil. If the duration of the impact load, t_p, is very short relative to the natural period of the machine foundation system (i.e. $t_p \ll T$), it can be assumed that the foundation goes through a free vibration triggered by an initial velocity, $\boldsymbol{v}_0$. For longer duration impact (i.e. greater than one-tenth of the natural period of the machine–foundation system), the response of the system is affected by the characteristics of the impact load. The impact velocity of the hammer can be calculated by

$$\boldsymbol{v}_0 = \sqrt{\frac{2{\cdot}2E_0}{m_0}} \tag{9}$$

where m_0 is the mass and E_0 is the maximum impact energy of the forging hammer, respectively. The velocity of the anvil after the impact, using the theory of conservation of momentum, is

$$\boldsymbol{v}_1 = \frac{m_0 v_0(1+e)}{m_0 + m_1} \tag{10}$$

where e is the coefficient of restitution whose magnitude may vary from 0·2 to 0·5. Thus, the impact force resulting from the collision between the hammer and anvil is given by

$$f_{(t)} = \begin{cases} m_1 \boldsymbol{v}_1/\tau & 60i/n < t \leqslant 60i/n + \tau \\ 0 & 60i/n + \tau \leqslant t + 60(i+1)/n \end{cases} \quad i = 0, 1, 2, \ldots \tag{11}$$

where τ is the collision time and n is the number of impacts per minute.

3.4. Method of analysis

The governing equilibrium equations of a two-mass system can be solved using the direct approach, by means of modal analysis, or numerically using a computer program. The direct approach involves substituting a particular solution into the basic differential equations and solving the resulting equation exactly using Kramer's rule. In the modal analysis, modal shapes and natural frequencies are also obtained as well as the amplitudes. The accuracy of model analysis deteriorates for highly damped systems (Novak and El-Hifnawy, 1983). On the other hand, computer-encoded numerical methods can easily be used to solve differential equations. The accuracy of the solution is highly noticeable for simple linear systems. In the current analysis, the equilibrium equations are solved using a central difference method with $\mathrm{d}t = T/500$ to obtain displacement and velocity time histories. The force transmitted through the mounting system can then be calculated as

$$f_{1(t)} = k_1(x_1 - x_2) + c_1(\dot{x}_1 - \dot{x}_2) \tag{12}$$

and the force transmitted through the soil is

$$f_{2(t)} = k_2 x_2 + c_2 \dot{x}_2 \tag{13}$$

4. PARAMETRIC STUDY

A comprehensive parametric study was conducted to investigate the effect of soil reinforcement on the foundation and anvil response and the force transmitted through the mounting system and foundation to the surroundings. The different mass ratios (m_1/m_2) are representatives of different configurations of isolated foundations. For small hammers where the anvil sits on an industrial pad and the frame is attached to the block, as shown in Figure 1(a), the mass ratio may vary from 0·11 to about 0·33. For larger presses, the entire press is supported by a mounting system that sits on a trough, as shown in Figure 1(b) and (c), and the mass ratio varies between 0·43 and 3. In the former case, using a pad is useful as long as pad stiffness, K_1, is less than about $0{\cdot}04K_2$. In the other cases, the mounting system can be useful as long as the stiffness K_1 is less than about $0{\cdot}30K_2$. The damping of the isolators is usually selected to be about 5 to 10% of the foundation damping – that is, $C_1/C_2 = 0{\cdot}05$–$0{\cdot}1$ (Chehab and El Naggar, 2003).

In order to establish the design charts for reinforced foundation with shock-absorbing system, the results are presented in a dimensionless form. The stiffness and damping of the foundation (K_2, C_2) are normalised by the stiffness and damping of the reinforced foundation (K_{2r}, C_{2r}). The vibration amplitudes of the anvil and the foundation block in the two-mass foundation system (A_1, A_2) are normalised by their vibration amplitudes in the two mass reinforced foundation systems (A_{1r}, A_{2r}). Similarly, the maximum amplitudes of the forces transmitted through the mounting system and the base in the two-mass foundation system ($\boldsymbol{F}_1$, $\boldsymbol{F}_2$) are normalised by the maximum amplitude of the forces transmitted through them in a two-mass reinforced foundation system ($\boldsymbol{F}_{1r}$, $\boldsymbol{F}_{2r}$).

5. RESULTS AND DISCUSSION

Figures 6 and 7 present the results for cases representative of small hammers where the anvil sits on an industrial pad and the frame is attached to the block directly. Figure 6 presents the variation of the anvil and foundation amplitudes of a foundation on RSF with the stiffness ratio K_{2r}/K_2 and a mass ratio $m_1/m_2 = 0{\cdot}11$, for different initial stiffness and damping ratio K_1/K_2 and C_1/C_2, respectively. Figure 6(a) shows that the anvil amplitude was slightly reduced, and Figure 6(b) shows that the foundation amplitude decreased significantly (up to 80%) as the stiffness of the RSF increased relative to the native soil. On the other hand, the force transmitted to the foundation increased slightly, and the force transmitted to the RSF either increased significantly (up to 60% for higher C_1/C_2) or decreased slightly (for low C_1/C_2) as the stiffness of the RSF increased, as can be noted from Figure 6(c) and (d), respectively. It is evident that the change in foundation responses and the forces transmitted through the system to the RSF as the stiffness ratio K_{2r}/K_2 increased was mainly dependent on the value of initial damping ratio C_1/C_2. Figure 7 shows the variation of responses of a hammer foundation and forces transmitted to the supporting RSF with the stiffness ratio K_{2r}/K_2 for $m_1/m_2 = 0{\cdot}33$. Similar observations can be made in this case. Comparing Figures 6 and 7, it can be seen that the beneficial effects of RFS increased and the adverse effects diminished as m_1/m_2 increased. The responses of the foundation system, the forces transmitted to foundation and the forces transmitted to RSF (for lower C_1/C_2) were improved slightly as the mass ratio m_1/m_2 increased. For higher C_1/C_2, the adverse effect of increasing the force transmitted to the RSF reduced significantly (from 60 to 30%) as the mass ratio m_1/m_2 increased.

Figures 8 to 11 present the variation of the vibration amplitudes of the foundation system and the forces transmitted to RSF with the stiffness ratio K_{2r}/K_2 for larger presses, where the entire press is supported by a mounting system that sits on a trough. In this case, the mass ratio m_1/m_2 typically varied between 0·43 and 3. Figure 8(a) shows that the anvil amplitude was slightly reduced, and Figure 8(b) shows that the foundation amplitude decreased significantly (up to 80%) as the stiffness of the RSF increased relative to the native soil. On the other hand, the force transmitted to the foundation increased (up to10%), and the force transmitted to the RSF either increased (up to 15% for higher K_1/K_2) or decreased slightly (for lower K_1/K_2) as the stiffness of the RSF increased, as can be noted from Figure 8(c) and (d), respectively. It is noted that the variation in the foundation responses and forces transmitted

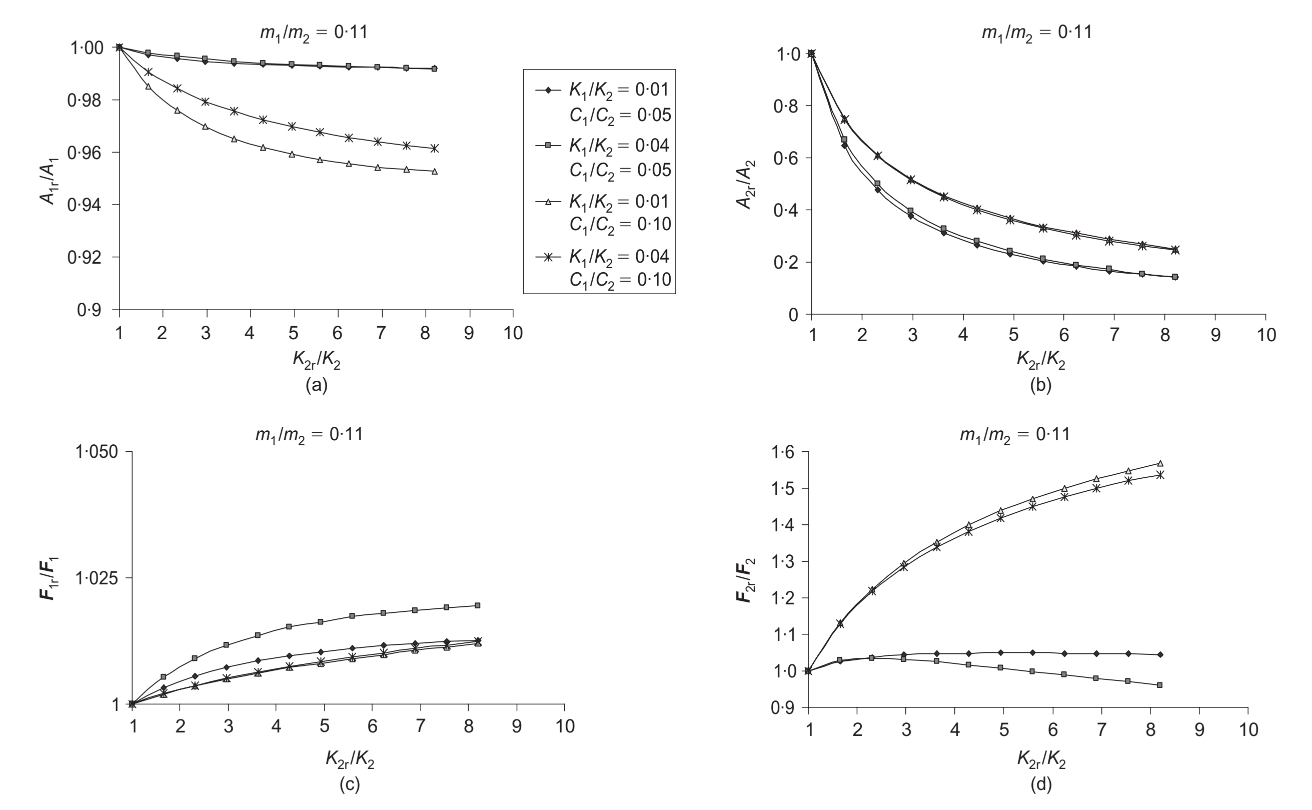


Figure 6. Variation of hammer foundation response and forces with stiffness of reinforced soil foundation (for $m_1/m_2 = 0{\cdot}11$): (a) anvil amplitude, A_1; (b) foundation amplitude, A_2; (c) force transmitted to foundation, $\boldsymbol{F}_1$; (d) force transmitted to RSF, $\boldsymbol{F}_2$

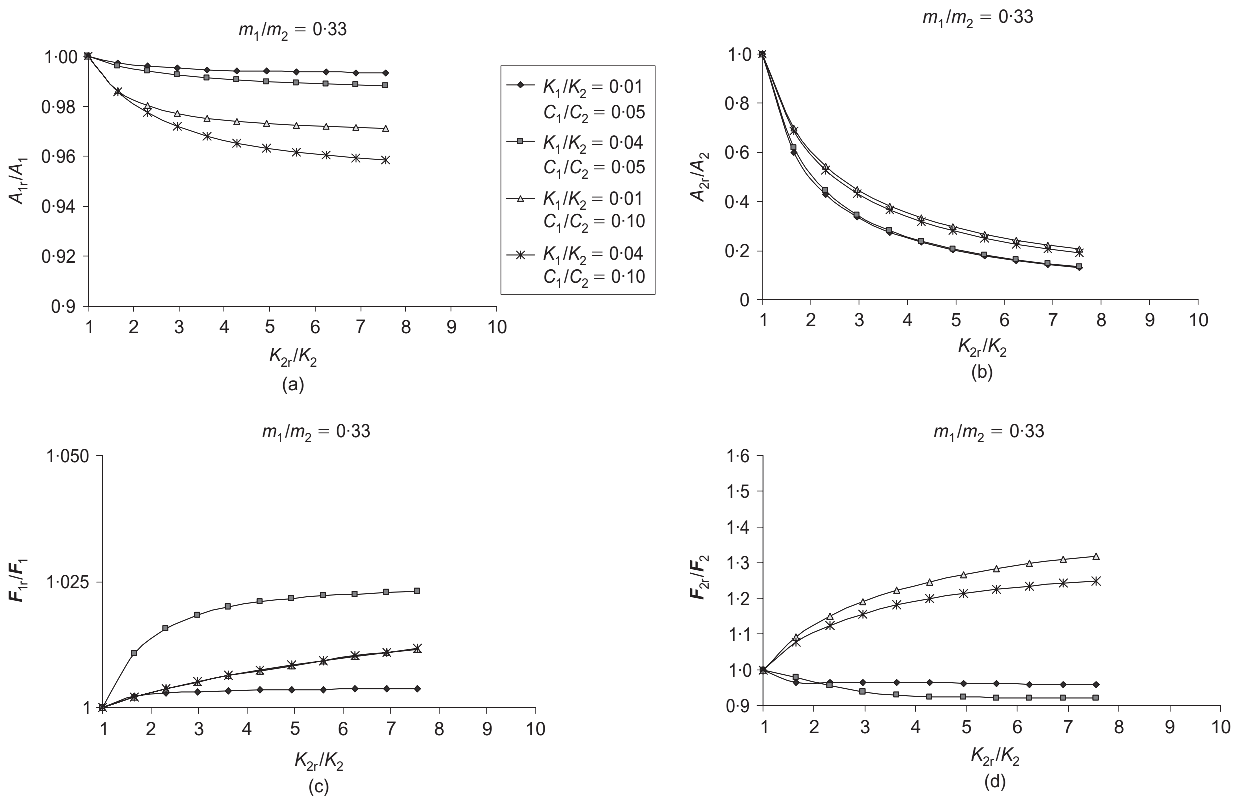


Figure 7. Variation of hammer foundation response and forces with stiffness of reinforced soil foundation (for $m_1/m_2 = 0.33$): (a) anvil amplitude, A_1; (b) foundation amplitude, A_2; (c) force transmitted to foundation, $\boldsymbol{F}_1$; (d) force transmitted to RSF, $\boldsymbol{F}_2$

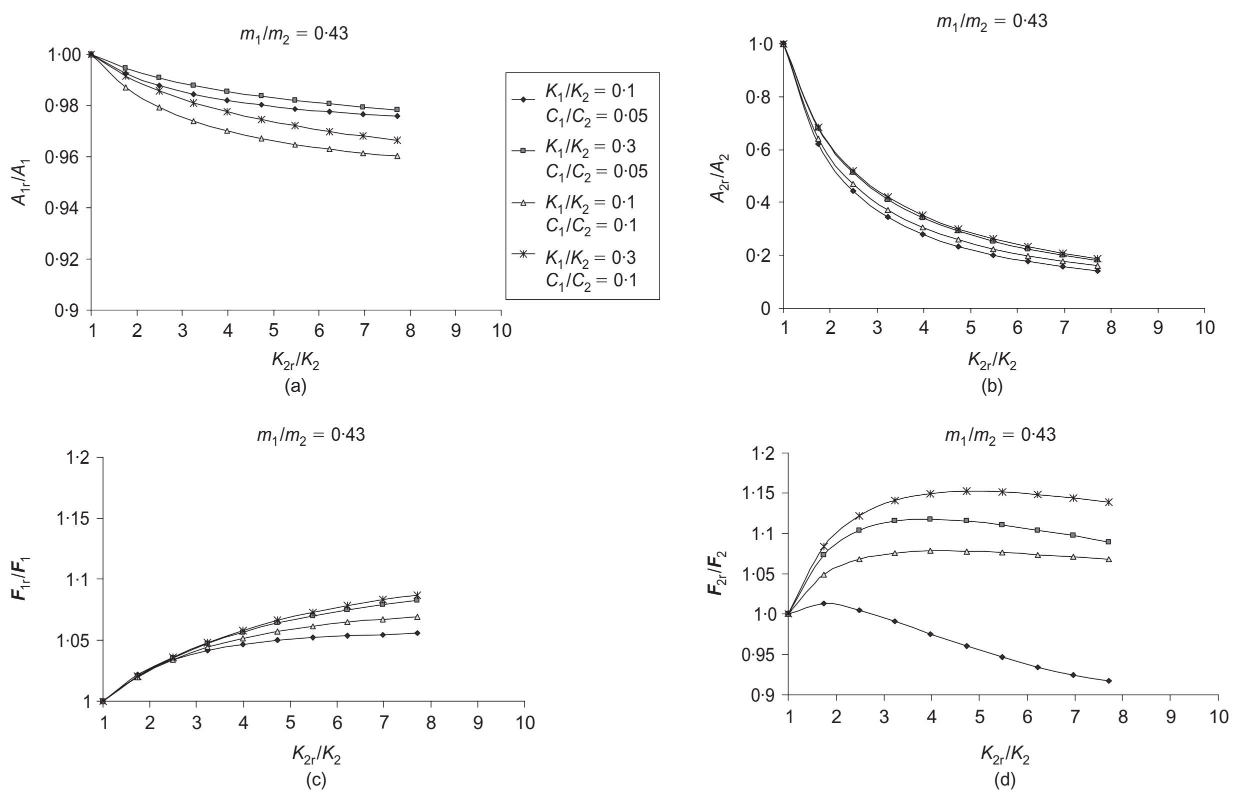


Figure 8. Variation of hammer foundation response and forces with stiffness of reinforced soil foundation (for $m_1/m_2 = 0.43$): (a) anvil amplitude, A_1; (b) foundation amplitude, A_2; (c) force transmitted to foundation, $\boldsymbol{F}_1$; (d) force transmitted to RSF, $\boldsymbol{F}_2$

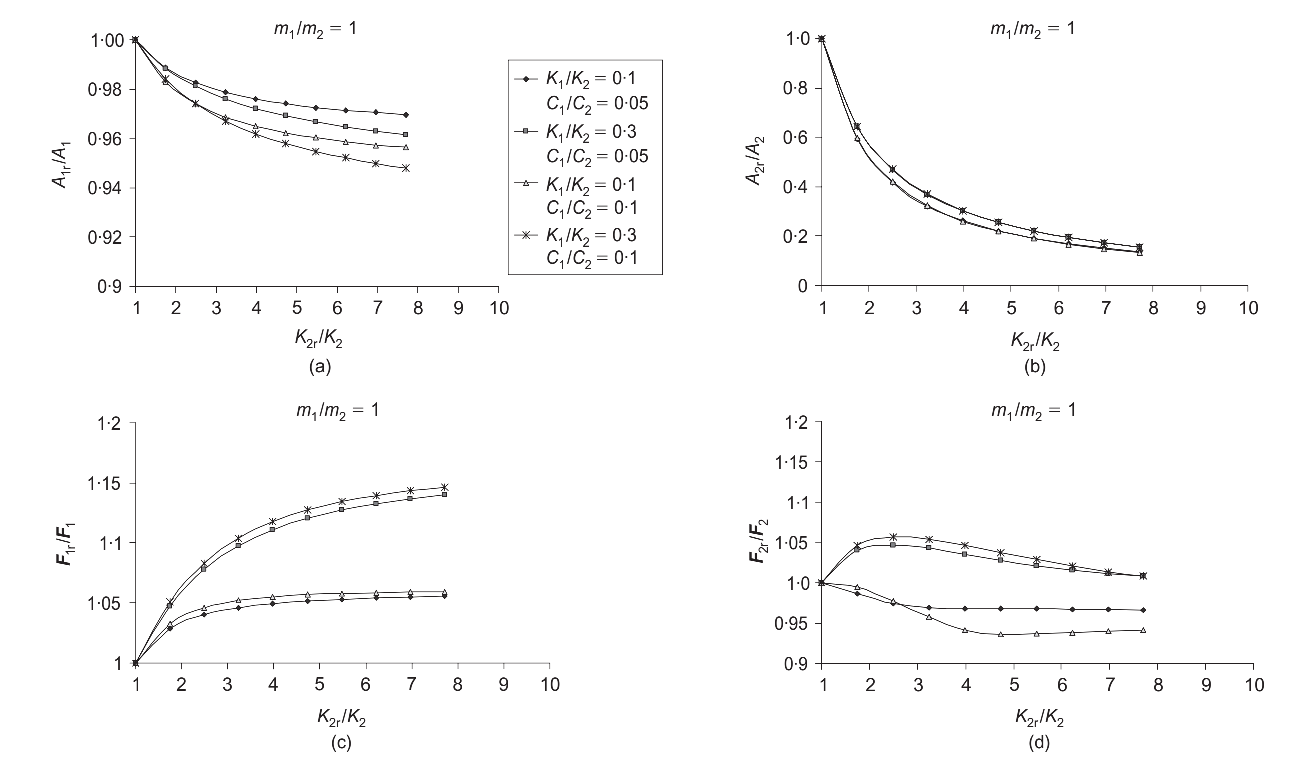


Figure 9. Variation of hammer foundation response and forces with stiffness of reinforced soil foundation (for $m_1/m_2 = 1$): (a) anvil amplitude, A_1; (b) foundation amplitude, A_2; (c) force transmitted to foundation, $\boldsymbol{F}_1$; (d) force transmitted to RSF, $\boldsymbol{F}_2$

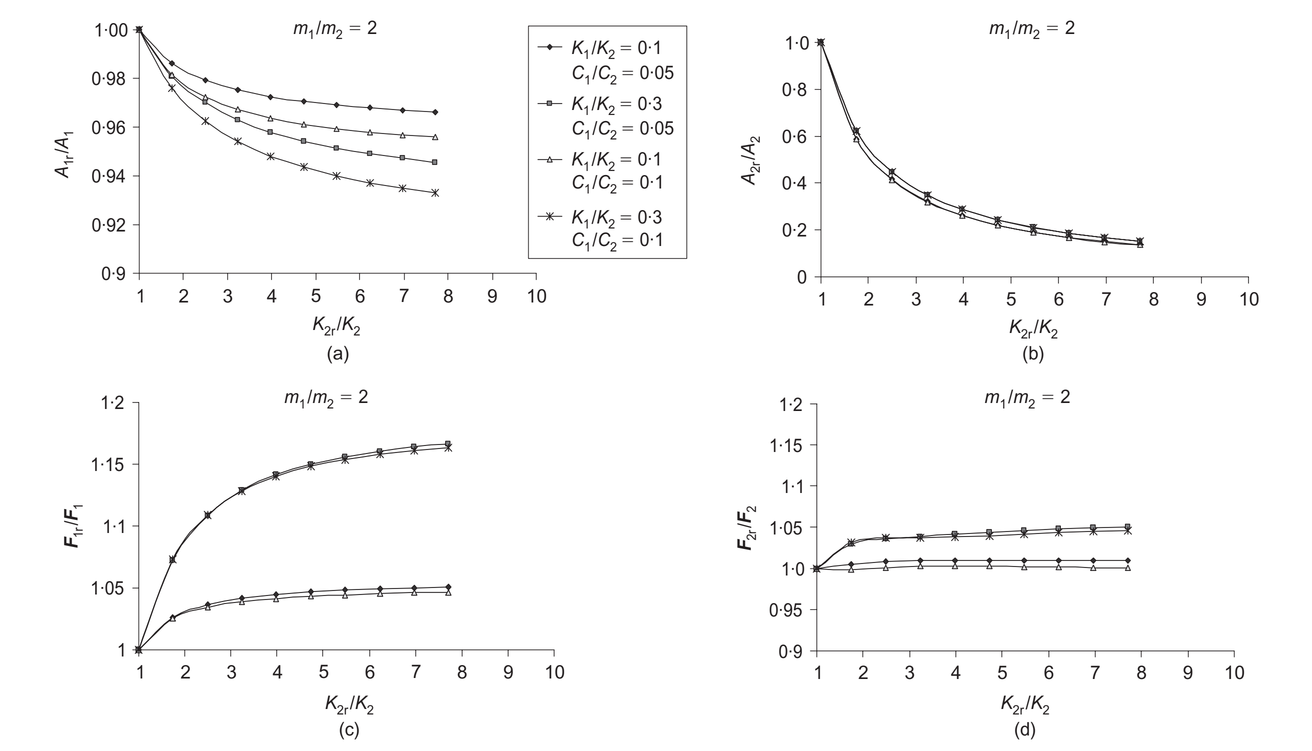


Figure 10. Variation of hammer foundation response and forces with stiffness of reinforced soil foundation (for $m_1/m_2 = 2$): (a) anvil amplitude, A_1; (b) foundation amplitude, A_2; (c) force transmitted to foundation, $\boldsymbol{F}_1$; (d) force transmitted to RSF, $\boldsymbol{F}_2$

through the system to the RSF with the stiffness ratio K_{2r}/K_2 depended mainly on the value of initial stiffness ratio K_1/K_2.

Figure 9 shows the variation of responses of a hammer foundation and forces transmitted to the supporting RSF with the stiffness ratio K_{2r}/K_2 for $m_1/m_2 = 1$. Figure 9(a) and (b) demonstrate trends similar to those noted in Figure 8(a) and (b), in terms of the variation of anvil and foundation

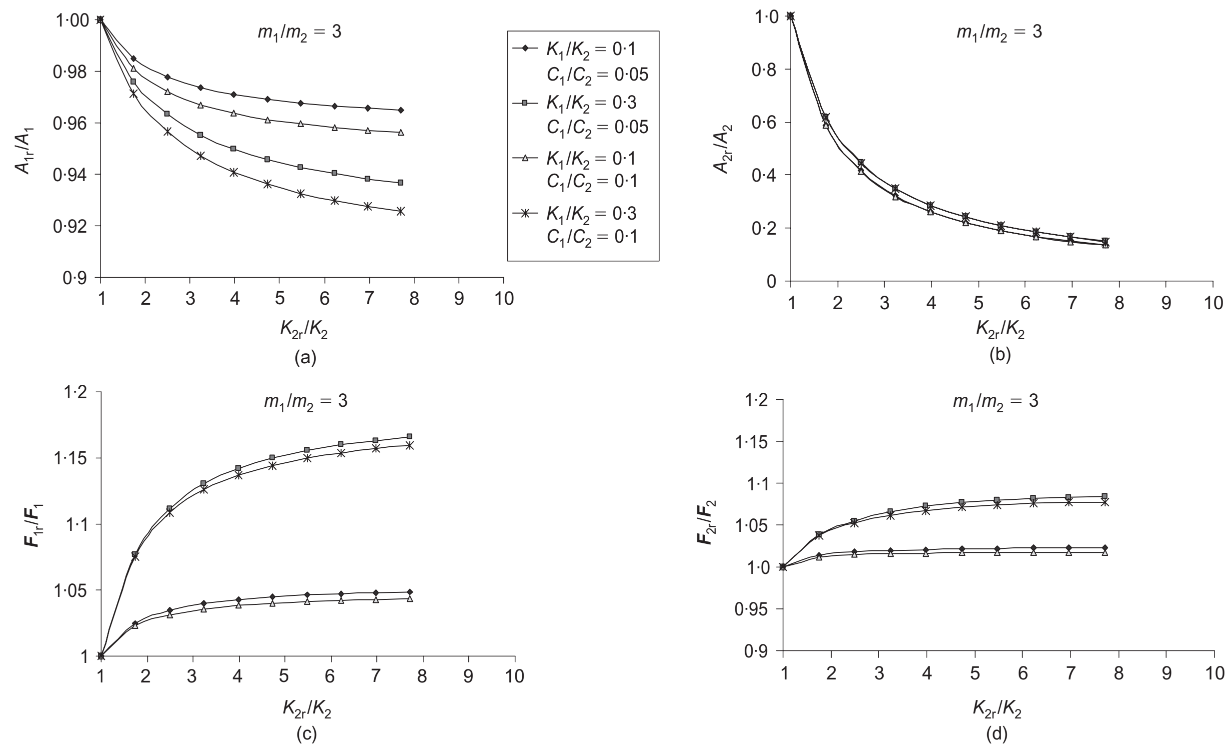


Figure 11. Variation of hammer foundation response and forces with stiffness of reinforced soil foundation (for $m_1/m_2 = 3$): (a) anvil amplitude, A_1; (b) foundation amplitude, A_2; (c) force transmitted to foundation, $\boldsymbol{F}_1$; (d) force transmitted to RSF, $\boldsymbol{F}_2$

amplitudes with the stiffness ratio K_{2r}/K_2 while the amplitude responses improved slightly. Although the force transmitted to the foundation may increase, as can be noted from Figure 9(c), its adverse effect either increased (for higher K_1/K_2) or slightly decreased (for lower K_1/K_2) in comparison with Figure 8(c). Furthermore, comparing Figures 8(d) and 9(d), it is clear that the adverse effect of increasing the force transmitted to RSF decreased (about 10% for higher K_1/K_2). Similar observations can be obtained from Figures 10 and 11 that show the variation of responses of the foundation system and forces transmitted to the RSF with the stiffness ratio K_{2r}/K_2 for $m_1/m_2 = 2$ and 3, respectively. Generally, the variations of the anvil and foundation amplitudes with the stiffness ratio K_{2r}/K_2 are likely to be the same as the mass ratio m_1/m_2 increases. Figures 10(c) and 11(c) show that for this range of the mass ratio (2 to 3) the force transmitted to the foundation remains almost unchanged. Figures 10(d) and 11(d) show similar trends for the variation of the force transmitted to the RSF whereas the force transmitted increased slightly as the mass ratio increased. However, by comparing Figures 10(d) and 11(d) with Figure 9(d), it is noted that increasing the mass ratio has a slight effect on the force transmitted to RSF.

6. DESIGN PROCEDURE

The design of foundations for vibrating equipment is usually governed by displacement considerations. The displacement of foundations subjected to impact load depends on the type and geometry of the foundations, the flexibility of the supporting ground and characteristics of pulse loading. The main objective of the design is to limit the response amplitudes of the foundation to the specified tolerance and minimise the maximum impact force transmissibility under design constraints, using the stiffness and damping coefficients of the isolator, mass of the foundation block and support area of soil as design variables.

The design of a machine foundation subjected to impact load involves a trial-and-error procedure. After establishing the soil profile and evaluating the soil properties required for the dynamic analysis (shear modulus, mass density, Poisson's ratio and material damping ratio), based on experience, the type and trial dimensions of the foundation should be selected. The maximum displacement and force transmitted to the soil of the one-mass foundation system (without a mounting system and soil reinforcement) under specific pulse loading can be computed easily (Chehab and El Naggar, 2004) and then compared with the performance criteria. If the response is not satisfactory, the force transmitted to the foundation and the vibration of the foundation (and consequently the disturbance to the surrounding medium) could be reduced by choosing the optimum configuration of mounting system and soil reinforcement using the foregoing design charts.

For small hammers, considering Figures 6 and 7, the designer should select the minimum stiffness of the mounting system, with its damping as small as possible. The soil reinforcement can then be designed to achieve satisfactory performance. The stiffness and damping of reinforced soil can be calculated as discussed in Section 3.2.2. The different parameters defining the geogrid-reinforcement can be varied to achieve the desired stiffness ratio K_{2r}/K_2 required to realise a certain reduction in the response. For example, stiffness ratio $K_{2r}/K_2 = 3$ yields a reduction in the foundation amplitudes of up to 60%. On the other hand, the force transmitted to the RSF remained unchanged as the lowest damping ratio was used. However, if the mounting system is chosen such that damping ratio $C_1/C_2 = 0{\cdot}1$, the force transmissibility increases up to 30%. The adverse effect of increasing the force transmitted to the RSF can be reduced significantly as the mass ratio m_1/m_2 increases. Thus, selecting an appropriate configuration of the mounting system or modifying the foundation dimensions to reach a higher mass ratio can be used as an effective option to reduce the force transmitted to the supporting RSF.

For the foundation of larger presses on RSF, the designer has to select the minimum stiffness of the mounting system while its damping ratio can have any value between 0·05 and 0·1 as can be stipulated from Figures 8 to 11. Similar to the case of small hammers, choosing the soil reinforcing scheme can provide a higher stiffness ratio K_{2r}/K_2 and using a proper configuration of the mounting system or modifying the foundation dimensions to achieve a desirable mass ratio. These two options together can be used effectively to satisfy the performance criteria.

7. SUMMARY AND CONCLUSIONS

The effect of soil reinforcement on the performance characteristics for different configurations of shock-absorbing foundations was investigated. Soil reinforcement can be used to increase the stiffness of the supporting medium. This increase can be designed to achieve a superior dynamic performance for shock-producing equipment when the mounting system alone can not achieve a satisfactory design. A parametric study was conducted and a set of charts was established as practical guidance for the design of soil-reinforcement schemes. It was shown that for small hammers, that the reinforced soil foundation can reduce the foundation response amplitude by up to 80%. For large hammers and presses, the reinforced soil foundation can be designed to reduce the foundation response by up to 60% of the case of no soil reinforcement.

ACKNOWLEDGEMENTS

The first author would like to acknowledge the fruitful discussions with Dr Nader Hataf who read a draft of the paper and offered valuable comments. The authors would also like to express gratitude to Professor Arsalan Ghahramani, who offered many useful suggestions.

REFERENCES

Al-Dobaissi HH (1990) *Footings on Reinforced Earth Subjected to Impact Loading*. MS thesis, University of Baghdad, Iraq.

Chehab A and El Naggar MH (2003) Design of efficient base isolation for hammers and presses. *Soil Dynamics and Earthquake Engineering* **23(2)**: 127–141.

Chehab A and El Naggar MH (2004) Response of block foundations to impact loads. *Journal of Sound and Vibration* **276(1–2)**: 293–310.

Chung W and Cascante G (2007) Experimental and numerical study of soil-reinforcement effects on the low-strain stiffness and bearing capacity of shallow foundations. *Geotechnical and Geological Engineering* **25(3)**: 265–281.

Das BM (1992) *Principle of Soil Dynamics*. PWS-KENT Publishing, Boston, MA, USA.

Das BM and Maji A (1994) Transient loading-related settlement of a square foundation on geogrid-reinforced sand. *Geotechnical and Geological Engineering* **12(4)**: 241–251.

Das BM, Maji A and Shin EC (1998) Foundation on geogrid-reinforced sand – effect of transient loading. *Geotextiles and Geomembranes* **16(3)**: 151–160.

El-Hifnawy L and Novak M (1984) Response of hammer foundations to pulse loading. *Soil Dynamics and Earthquake Engineering* **3(3)**: 124–132.

El Naggar MH and Abdel-Meguid M (1997) Effect of soil treatment on the dynamic response of piles. *Proceedings of the 50th Canadian Geotechnical Conference, Ottawa*, 798–805.

El Naggar MH and Wei J (1997) Dynamic response of machine foundation in improved soils. *Proceedings of the 50th Canadian Geotechnical Conference, Ottawa*, 356–363.

Ghosh C and Madhav MR (1994) Reinforced granular fill-soft soil system: confinement effect. *Geotextiles and Georaembranes* **13(11)**: 727–741.

Guido VA, Knueppel JP and Sweeney MA (1987) Plate load test on geogrid-reinforced earth slabs. *Proceedings of Geosynthetics '87, New Orleans*, 216–225.

Heydari M, Ghahramani A and El Naggar MH (2008) Effect of soil reinforcement on the response of vertically vibrating foundations using different isolation systems. *Proceedings of the 4th European Geosynthetics Conference, Edinburgh*, September 7–10, Paper 108.

Jones CJFP (1985) *Earth Reinforcement and Soil Structures*. Butterworth, London.

Kavazanjian Jr E and Matasovic N (1995) *Seismic Analysis of Solid Waste Landfill, Geoenvironment 2000*. ASCE, New York, NY, USA, Geotechnical Special Publication No. 46.

Li J and Ding DW (2002) Nonlinear elastic behavior of fiber-reinforced soil under cyclic loading. *Soil Dynamics and Earthquake Engineering* **22(9–12)**: 977–983.

Montanelli F and Recalcati P (2003) Geogrid reinforced railways embankments: design concepts and experimental test results. *Proceedings of the IABSE Symposium, Antwerp*, 212–213.

Novak M (1974) Dynamic stiffness and damping of piles. *Canadian Geotechnical Journal* **11(4)**: 574–598.

Novak M (1983) Foundations for shock-producing machines. *Canadian Geotechnical Journal* **20(1)**: 141–158.

Novak M and Beredugo YO (1972) Vertical vibration of embedded footings. *Journal of the Soil Mechanics and Foundations Division, ASCE* **98(SM12)**: 1291–1310.

Novak M and El-Hifnawy L (1983) Vibration of hammer foundations. *Soil Dynamics and Earthquake Engineering* **2(1)**: 43–53.

Omar MT, Das BM, Puri VK and Yen SC (1993) Ultimate bearing capacity of shallow foundations on sand with geogrid reinforcement. *Canadian Geotechnical Journal* **30(3)**: 545–549.

Prakash S (1981) *Soil Dynamics*. McGraw-Hill, New York, NY, USA.

Shin EC, Das BM, Lee ES and Atalar C (2002) Bearing capacity of strip foundation on geogrid-reinforced sand. *Geotechnical and Geological Engineering* **20(2)**: 169–180.

Shuwang Y, Xingiang Z and Run L (2004) An FE analysis of reinforced subgrade under automobile loading. *Acta Mechanica Sinica* **20(1)**: 1–8.

Uchimura T, Nguyen CA and Mizuhashi M (2006) Effects of reinforcement stiffness on deformation of reinforced soil structures under cyclic and sustained load. *Proceedings of the*

8th International Conference on Geosynthetics, Yokohama*, 1.

Wang G and Dong Z (2006) Design optimization of low impact transmission foundation for forging hammers. *Engineering Computations: International Journal for Computer-Aided Engineering and Software* **23(2)**: 166–186.

Yegian MK and Harb JN (1995) *Slip Displacement of Geosynthetic Systems under Dynamic Excitation*. ASCE, New York, Geotechnical Special Publication No. 54.

Yegian MK, Harb JN and Kadakal U (1998) Dynamic response analysis procedure for landfills with geosynthetic liners. *Journal of Geotechnical Enginering, ASCE* **124(10)**: 1027–1033.

What do you think?

To discuss this paper, please email up to 500 words to the editor at journals@ice.org.uk. Your contribution will be forwarded to the author(s) for a reply and, if considered appropriate by the editorial panel, will be published as discussion in a future issue of the journal.

Proceedings journals rely entirely on contributions sent in by civil engineering professionals, academics and students. Papers should be 2000–5000 words long (briefing papers should be 1000–2000 words long), with adequate illustrations and references. You can submit your paper online via www.icevirtuallibrary.com/content/journals, where you will also find detailed author guidelines.