Case Study of Dynamic Responses of a Single Pile Foundation Installed in Coal Ash Landfills using Effective Stress Analysis and EQWEAP

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ABSTRACT: Coal ash continues to be a major production of energy generated material and its use are predicted to increase. The disposal of coal ash has become an important issue when not only chemical but also physical and engineering characteristic of the coal ash is necessary to be researched. The large quantity of disposal of coal ash is even more complicated to handle. This paper is focused on researching dynamic behaviors of coal ash soils obtained in a landfilled field in north Taiwan and also the dynamic interaction of a single pile foundation sitting in the landfills. An effective stress analysis, firstly, was adopted for examining seismic response of 4 m height single pile with 10 m depth into the ground, of which soil parameters in the constitutive model was confirmed basis of stress path and stress- strain relationship of the coal ash soils. Then, the pile response was researched again by EQWEAP. This paper found out that 1) when the pore water pressure reached 40 kpa at around 5sec, the coal ash landfill liquefied, 2) the movement of pile foundation became larger after 5sec, 3) the nonlinearity of the soils resulted in reducing responded horizontal acceleration of the soils at different depth after 5sec. It also showed that both numerical codes were applicable of reproducing pile responses in this an extreme weak coal ash in this study.

KEYWORDS: Dynamic responses of single pile foundation, Soil liquefaction, Coal ash, Three dimensional effective stress analysis, EQWEAP

1. INTRODUCTION

Amount of coal ash to be disposed of as a waste at utility disposal sites is increased year by year. Generally speaking, the difference between fly ash and bottom ash is that fly ash is finer ash particles suspended in the boiler furnace during coal combustion and bottom ash is consisted of relative coarser particles settled at the bottom of the boiler furnace during coal combustion. Coal ash continues to be a major production of energy generated material and its use are predicted to increase. The disposal of coal ash has become an important issue when not only chemical but also physical and engineering characteristic of the coal ash is necessary to be researched. The large quantity of disposal of coal ash is even more complicated to handle. The use of coal ash in construction projects is promising to replace the traditional materials if the environment and geotechnical engineering problem is controlled well.

This paper focuses on the dynamic characteristics of fly ash and bottom ash mixtures and the dynamic responses of a single pile installed in the coal ash landfill. This could be an initial step to probe into the factors affecting the dynamic behaviour of coal ash and further in various engineering applications.

In general, coal ashes can be classified into coal source types (bituminous or anthracite), coal ash source (segregated or unsegregated fly and bottom ash), coal ash "gradation" (classification based upon the sizes of the ash particles), and other coal ash properties such as pH (relative "acidity" or "alkalinity") relevant to the proposed usage. Engineering methods to determine gradation generally classify a soil or coal ash by the percentages of particles that can pass through standardized sieve opening sizes via comparison to a USDA Soil Triangle Classification Chart. Coal ash generally contains approximately 60 to 70 percent silt, and 30 to 40 percent sand size particles depending on the characteristics of the fuel burned by the plant. The coal ash classification is normally that of a silt loam. In our study field, the specific gravity coal ash is 14 kN/m^3 and with around 60 percent silt.

Some previous studies have aroused attention on use of engineering characteristics of coal ash soils [18, 19, 20, 24, 31, 32], particularly attention should be paid on its resistance against liquefaction. There are amount of papers and good experiences regarding to dynamic behavior of structure on liquefiable soils [1-12], however, very limited in coal ash soils.

2. DYNAMIC CHARACTERISTICS OF COAL ASH

2.1 Performance of the constitutive model

In a numerical simulation, an appropriate approach for obtaining reliable parameters for soil model was necessary. In the soil model used in the present study, parameters such as e_0, λ, κ, OCR^*, M^*_m, M^*_f and $G_0/\sigma_{m'}$ could be directly determined by physical property tests and undrained monotonic shear tests. The rest parameters could be determined by physical property tests and un- drained monotonic and cyclic shear tests.

Physical experimental data on coal ash soils (Dr=50%, 60% and 70%) were done in previous research [24]. Through the previous papers, the authors could obtain almost all the needed parameters introduced above. The application of the parameters of sandy soils had been discussed and their workability was confirmed [9 and12]. Figure 1 demonstrated the liquefaction strength of the simulated coal ash and the some sandy soils. The remaining parameters including 4 kinematic parameters that could not directly obtain from the previous research [24] require curve fitting, the details of curve fitting principle could be referred to Oka et al. [7]. The soil parameters for coal ash with relative density of 50% used in the simulations are listed in Table 1. And the simulated and experimental performance of the coal ash soils could be seen in Figure 2.

As shown in Figure 1, the liquefaction strength of the coal ash soils (Dr=50%) with various cycles agreed well with the liquefaction strength curve of sandy soils in the similar relative density. It should be kept in mind that even the liquefaction strength was close, soil behaviours could be totally different but still, the liquefaction strength could be a good reference for evaluating impact from liquefied soil on structures. Therefore, ones need the stress- strain relationship and stress path of the coal ash soils. In this study, the authors highlighted the responses of single pile due to liquefied soils in the ground consisted of coal ash soils.

Comparing with the parameter of coal ash soils listed in Table 1 and Toyoura sand, some clear differences could be seen that the compressibility of the coal ash soils in this case study is much higher and the specific gravity is much lower.

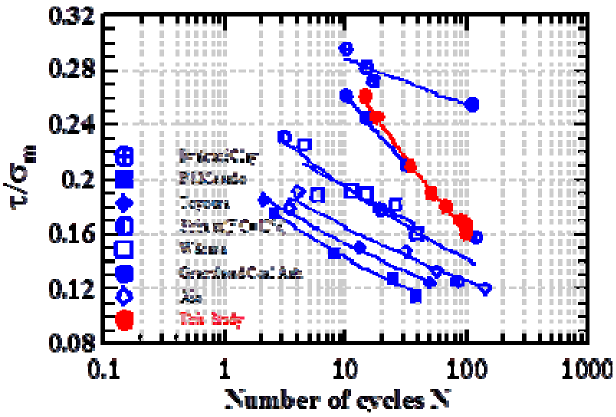

Figure 1 Liquefaction strength of coal ash and different soils

Table 1 The parameters of coal ash soils in FE analysis

Name of soil profile	Dr=50%
Density ρ ($t{\cdot}m^{-3}$)	1.400
Coefficient of permeability K ($m{\cdot}s^{-1}$)	1.4×10-6
Void Ratio e_0	2.170
Compression Index λ	0.0430
Swelling index κ	0.00430
Normalized Shear Modulus $G_0/\sigma_{m'}$	550
Stress Ratio at Maximum Compression M_m^*	1.114
Stress Ratio of Failure State M_f^*	1.560

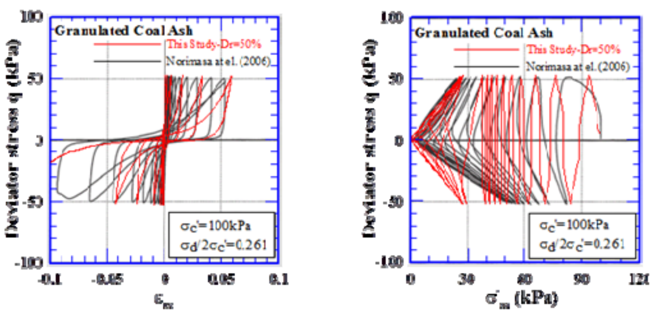

Figure 2 Performance of the simulated coal ash versus experimental work

3. NUMERICAL METHOD

Lu et al. [9-12] has applied the following numerical schemes on analyzing liquefaction related projects such as dynamic behaviors of single pile foundation, group pile foundation, grounds consisted of sandy soils with various relative densities, spreading liquefied soils and settlement of shallow foundation etc. The results showed the scheme is reliable.

On the other hand, seismic responses of the piles could be analyzed using the time-dependent Winkler type foundation model, whereas a simplified two-step procedure EQWEAP has been suggested by Cheng [13][14]. In such analysis, the free-field ground motions are obtained first and then applied to the pile for the solutions, and the discrete wave equations are used to solve for the pile displacements. EQWEAP modelling was shown in good agreements with the FE solutions. To model the soil liquefaction and/or liquefaction induced lateral spreading, a number of alternative models have also been suggested.

In FE analysis, a soil-water coupled problem was formulated based on a u-p formulation [7]. The finite element method (FEM) was used for the spatial discretization of the equilibrium equation while the finite difference method (FDM) was used for the spatial discretization of the pore water pressure in the continuity equation. Oka et al. [7] verified the accuracy of the proposed numerical method through a comparison of numerical results and analytical solutions for transient response of saturated porous solids. The governing equations were formulated by the following assumptions; 1) the infinitesimal strain, 2) the smooth distribution of porosity in the soil, 3) the small relative acceleration of the fluid phase to that of the solid phase compared with the acceleration of the solid phase and 4) incompressible grain particles in the soil. The equilibrium equation for the mixture was derived as follows:

$$\rho \ddot{u}^s{}_i = \sigma_{ij,j} + \rho b_i \qquad (1)$$

in which ρ is the total density, $\ddot{u}^s i$ is the acceleration of the solid phase, σ_{ij} is the total stress tensor and b_i is the body force vector. The continuity equation is derived as follows:
The continuity equation is written as

$$\rho^f \ddot{u}^s_{i,i} - p_{,ii} - \frac{\gamma_w}{k}\dot{\varepsilon}^s_{ii} + \frac{n\gamma_w}{kK^f}\dot{p} = 0 \qquad (2)$$

where ρ^f is the density of fluid, p is the pore water pressure, γ_w is the unit weight of the fluid, k is the coefficient of permeability, ε^S_{ii} is the volumetric strain of the solid phase, n is porosity and K^f is he bulk modulus of the fluid phase. The Newmark implicit method was used for time integration.

The constitutive equation used for sand was a cyclic elasto-plastic model. The constitutive equation was formulated by the following assumptions; 1) the infinitesimal strain, 2) the elasto-plastic theory, 3) the non-associated flow rule, 4) the concept of the overconsolidated boundary surface and 5) the non-linear kinematic hardening rule. Oka et al [8] discussed the applicability of the constitutive model for the cyclic undrained behavior of sand through a comparison of numerical results and hollow cylindrical torsional shear tests. The model succeeded in reproducing the experimental results well under various stress conditions such as isotropic and anisotropic consolidated conditions, with and without the initial shear stress conditions and the principal stress axis rotation.

3.1 FEM model

Figure 3 showed the configuration of the single pile foundation in coal ash landfills. Width of the system is 20 m, length is 55.2 m and height is 10 m. The pile is lifted 4 m above ground with 10m
The pile is lifted 4 m above ground with 10m undergoing to the bottom as an end bearing pile. The soils were modeled with 8-node isoparametric solid elements. It contained 1386 nodes and 1000 elements in the numerical mesh.

The authors used the cyclic elasto-plastic model to represent all soil layers. The parameters for each of soil layers are shown in Table 1. The elements below the water table were treated as fully saturated elements with DOF (Degree of Freedom) of pore water pressure. The pile was modeled by linear elastic beam elements. The diameter of the pile was 60 cm and the bending stiffness (EI) of was 8000 kNm^2. The pile length is 14m with 4m elevated above ground. The loaded mass on pile head is 80 ton. And the pile tip reached the bottom of the ground. No slip in the horizontal direction between the pile and soil was assumed.

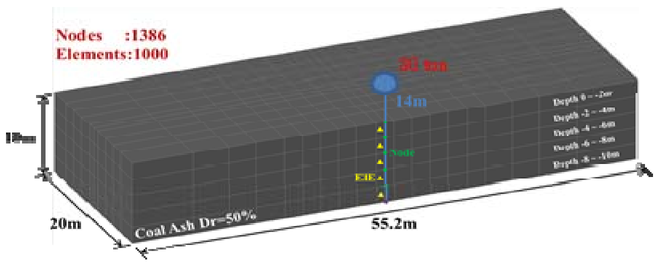

Figure 3 Configuration of the numerical system of FE analysis

3.2 Boundary conditions

For the boundary conditions, the bottom of the mesh was set to be rigid and all lateral boundaries were set to be equal-displacement to avoid unnecessary echo-vibration and to simulate the side boundary conditions of the laminar box. The input acceleration was set at the rigid bottom boundary. The seismic wave used was sine wave with 1Hz of frequency, and 100 gal and 200 gal of magnitude, namely A1F1 and A2F1. As to the drainage boundary condition, the lateral and bottom boundaries were assumed to be impermeable while the water table was permeable.

3.3 Initial conditions

The initial stress state was computed by the static analysis in which the two degree angle was neglected. Then, the dynamic analysis with keeping internal variables was conducted.

3.4 Other numerical conditions

A time integration step of 0.01 second was adopted to ensure the numerical stability. The hysteresis damping of the constitutive model was used and the assumption that the Rayleigh damping was proportional to the initial stiffness was used in order to describe the damping especially in the high frequency domain. β and γ in the Newmark method were set to be 0.3025 and 0.6 to ensure the numerical stability.

4. ANALYZED RESULTS AND DISCUSSIONS

4.1 Excess pore water pressure by FE model

Figure 4 showed the responded excess pore water pressure at several time. It could be seen that the excess pore water pressure built up and the loose ground started to liquefy at 5 sec. Because coal ash soils are very compressive, therefore, negative pore water pressure developed at 1.5 sec and also excess pore water pressure developed even slower in 200 gal shake than in 100 gal shake.

It should be noted that the initial effective stress of the 10m depth is 40 kPa because the submerged unit weight of the coal ash is 4kN/m^3 in this case study. This is quite different from the general soils in practice and it reminds us the key factor for liquefaction counter measures implemented in coal ash soils.

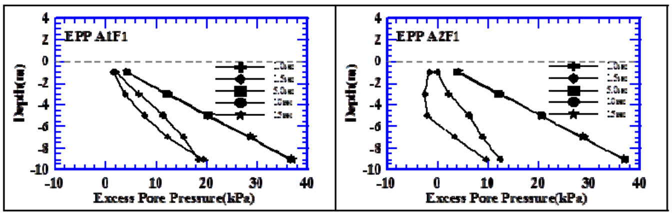

Figure 4 Distribution of the excess pore water pressure at different times due to FE analysis

4.2 Movement of pile top by FE model and EQWEAP

The movement of pile top responded could show interaction between structure and soils shown in Figure 5. Before 3 sec, the movement of pile top in 100 gal and 200 gal shake was very similar to each other, because the ground was still relative hard and the SSI, therefore, controlled by solid ground. After 5 sec, the movement between two pile tops showed great difference obviously due to the built up excess pore water pressure. The final deformation of the pile top in 200 gal shake was much more than 2 times to 100 gal shake because of the non- linearity of coal ash soils in the analysis. It reminded that the SSI shall be considered by an appropriate soil model while dealing with structures in liquefiable soils.

In comparison with other type of solution, in which validation of the analyses can be reached, one-dimensional pile responses under the excitations were modelled using a discrete wave-equation analysis called EQWEAP (Chang et al., 2010 and 2014). The EQWEAP analysis is based on lumped mass analysis to obtain the free-field ground responses. Once the site responses were obtained, the corresponding pile responses can be computed solving the wave equations of the pile segments. Using the EQWEAP analysis, the authors were able to find the similar solutions of the pile response. Note that although the EQWEAP analysis provide one-dimensional time-dependent responses of a single pile subjected to dynamic loadings, the solution can be treated as a simplified one for two-dimensional structural problem. Table 2 list the soil parameters used in EQWEAP analysis. Figures 6 showed respectively the surface ground responses and the pile head displacement functions plotted with time. The single pile responses above the ground were obtained solving a SDOF structural system accelerated by the ground motions whereas the ground was under the same steady-state accelerations with different peaks. It can be found that the motions are mainly controlled by the ground. A clear amplification phenomenon is able to reproduce using different numerical analyses. It should be point out that the difference between the numerical analysis is because that the soil material in FE analysis is controlled by a more complicated model, therefore, soil strength would be varied due to many various stress states. Further calibrations could be done to minimize their differences, however it is pointless at this moment for a preliminary study. Such comparison depicts that these numerical solutions are rational and can be used for reliable predictions.

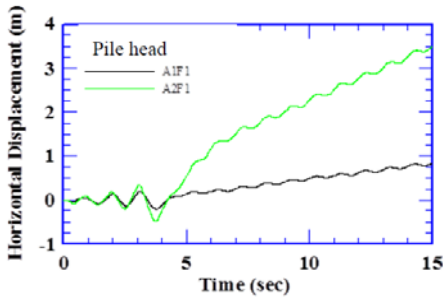

Figure 5 Movement of pile head due to FE analysis

Table 2 Soil parameters used in EQWEAP analysis

Name of soil profile	Dr=50%
Unit weight (kN/m^3)	14
Angle of friction ψ	36
SPT-N	20
Damping (%)	5
Cohesion (kg/cm^2)	2

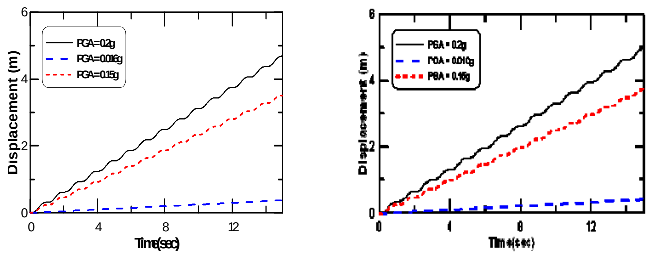

Figure 6 Movement of ground surface and pile head by EQWEAP

4.3 Time history of acceleration responses by FE model

Figure 7 showed the responded acceleration of pile and ground where is 5m away to the pile at the same depths and of pile top. It revealed that the ground responses decreases significantly before 5sec, because of the pore water pressure development as mentioned in the previous section. In the meanwhile, the pile responses reduce greatly due to the fact that the ground significantly influences the responses of pile. It also showed that the pile at deeper depths have a greater responses to seismic activity than at lower depths and is magnified again at segments above ground.

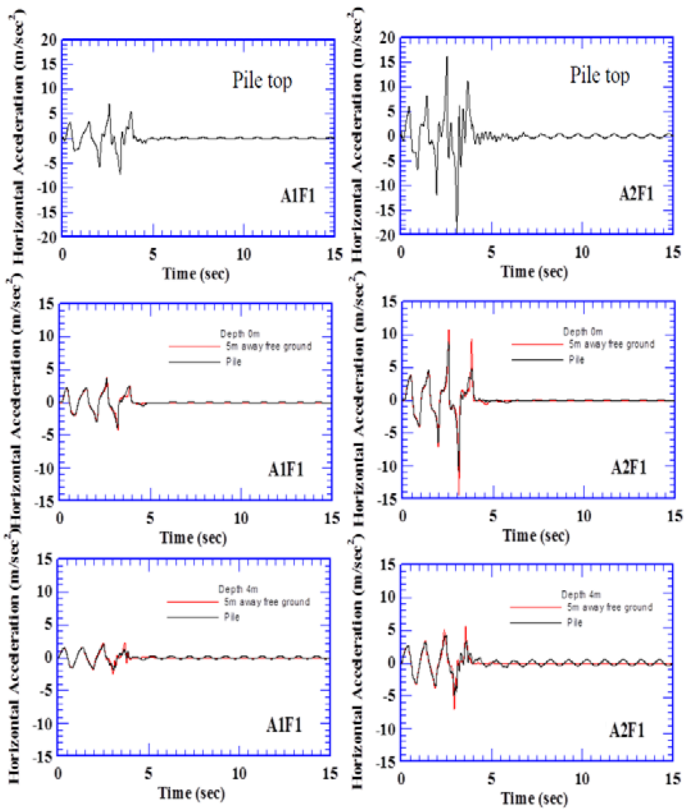

Figure 7 Time history of acceleration responses at different depth of pile and ground by FE analysis

4.4 Acceleration distribution by FE model

Acceleration of piles at different depth was shown in Figure 8. The acceleration at higher part of pile was larger at 1 sec and 1.5 sec. The ground at these moments played a role as a relative strong medium for wave propagation and amplified the accelerations, on the other hand, played as fluid to shelter water propagation after liquefied after 5 sec. It could be told that while the liquefied soils confined the single pile, the waves could not propagate through the pile because, still, ground controlled the responses of pile.

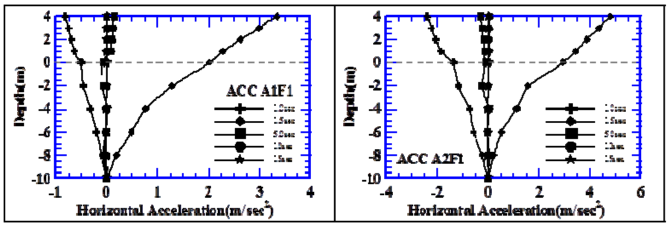

Figure 8 Distribution of acceleration at different depth of pile by FE analysis

4.5 Deformation of the ground and single pile foundation system by FE model

Figure 9 showed the deformation of the ground- pile system at the end of shake, it goes without saying that the system deformed larger in 200 gal shake than in 100 gal. The color in the mesh represented the excess pore water pressure, and it showed that the whole ground liquefied at t= 15 sec.

It also could be seen that no matter the shake was 100 gal or 200 gal, the boundary of the ground- pile system was not disturbed which also showed to the readers that adopted boundary methodology is reliable in this paper.

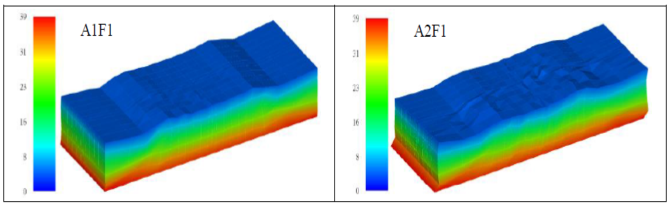

Figure 9 Deformation of the ground and single pile system and its excess pore water pressure distribution at t=15sec by FE analysis

5. CONCLUSION

This paper focuses on studying dynamic characteristics of fly ash and bottom ash mixtures and the dynamic responses of a single pile installed in the coal ash landfill by numerical analysis. The authors conducted an effective stress analysis on the ground – single pile foundation system excited by sine wave shakes in 100 gal and 200 gal with 1Hz and EQWEAP analysis is used to confirm the pile responses.

The results showed that

1) The coal ash soils in this case study showed more compressibility over general sandy soils in the same relative density, therefore, this characteristic enhance its liquefaction resistance.
2) The specific gravity of coal ash soils in this case study was much smaller than general sandy soils in the same relative density, therefore, this characteristic reduce its liquefaction resistance significantly.
3) Combing the above characteristics, the ground consisted of the coal ash soils in this case study, was shake to liquefy slower than Toyoura sand in the same relative density, and the liquefied ground heaved more obvious.
4) The single pile foundation was moved much greater in 200 gal shake than in 100 gal, which once again reminded us the use of nonlinearity soil model was essential in this type of analysis.
5) The wave propagation of pile, the starting of large deformation of ground and the deformation of pile all were responded to the liquefaction extent of ground. This raises attention on dealing with the geotechnical problems to engineers and researchers.
6) Both numerical tools could be able to provide the pile responses. In which, EQWEAP analysis could provide a fast, simple and reliable assistance and FE analysis could give an information of the SSI in more details.

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