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Simple Mathematical Approach to Simulate Granular Fill Behavior under Dynamic Compaction

Dr. Ahmed M. Ebid,
Lecturer, Str. Dpt., Faculty of Engineering & Technology, Future University, Cairo, Egypt

Abstract*—improving soil parameters using dynamic compaction of was intensively studied by many researchers since 1980's. Earlier researchers depended on statistical analysis of many case studies and soil dynamic principals to develop empirical formula used in designing dynamic compaction procedure. Recent researchers used different finite element models to describe the behavior of soil under dynamic compaction; those models varied between 1-D simple model and up to 3-D sophisticated ones. The aim of this research is to introduce a simple mathematical approach to simulate ground deformations and soil parameters improvement due to dynamic compaction. The proposed approach consists of two equations, the 1st one used to calculate the ground settlement due to one temper drop, the 2nd one used to calculate the updated soil parameters due to the ground settlement from the previous drop. By applying the two equations successively, both ground settlement and soil parameters improvement could be calculated after each tamper drop. The proposed approach was applied on four case studies and its results were so close to measured ones. The proposed approach could be used in designing or testing the dynamic compaction procedures and also in monitoring the quality of execution by comparing the measured settlement after each drop with calculated one.

***Index Terms*—dynamic compaction, Granular fill, Soil improvement, Simple approach**

I. INTRODUCTION

Dynamic compaction (DC) is one of the earliest ground improvement techniques. It depends on rearrange the soil particles using dynamic energy produced by dropping a weight (tamper) from a certain height, (Refer to figure (1)). Ground response to dynamic compaction was intensively studied by many researchers since 1980's. Earlier researchers (1980's-1990's) such as Mayne[1],[2] and Chow[3],[4] used principals of soil dynamics to study the response of ground under dynamic compaction. Mayne et al, developed number of empirical formulas based on collected data from 110 sites, those formulas describe the relation between peak particle velocity (PPV), Depth of Influence (D), tamper weight (W), tamper diameter (B) and dropping height (H). He also derived a formula to calculate the maximum stresses in soil due to (DC) at any depth (Z) below the tamper function in shear wave velocity (Vs) as follows:

$$PPV = 7\left(\frac{\sqrt{WH}}{D}\right)^{1.4}, \quad D_{max} = n\sqrt{WH}, \quad \sigma_{z\,max} = \frac{V_s\sqrt{WHB}}{(B+Z)^2}$$

Where (W) in tons, (H,D,B,Z)in meters, (PPV) in cm/sec , (Vs) ranged between 200-300 m/s and (n) empirical factor depends on soil type and ranged between 0.3-0.8, refer to table (1). Chow et al developed a procedure to estimate the depth of the crater and the enhancement in soil strength based on wave equation similar to one that used to predict the penetration of driven piles. Also, he studied the soil strength enhancement around the tamper. He approved that the enhancement decreases logarithmically with the distance to center of tamper and the enhancement vanish at distance equals to 3.5 tamper diameter.

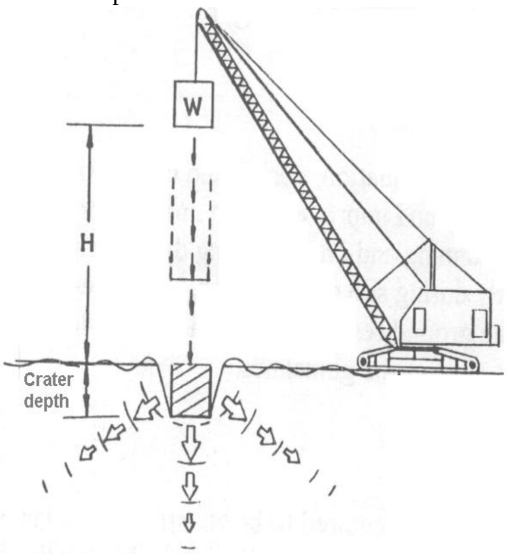

Fig (1): Dynamic Compaction, after Lukas[9]

Recent researchers (2000's-2010's) such as Pan[5], Lee[6] and Xie[7] used more advanced and sophisticated techniques to study the response of ground under dynamic compaction.

Pan used a 2D ABAQUS finite element model to simulate the impact of the tamper with the ground, he visualized the impact wave propagation through the soil and calculate the dynamic load value, peak particle velocity and depth of crater, however, he didn't estimate the enhancement in soil strength due to (DC). Lee and Gu used a 2D non-graphical CRISDYN finite element model to study the ground response under (DC). Using his model, he carried out parametric studies for the relations between tamper weights, diameter, dropping height and grid spacing on depth of influence, crater depth and vertical stresses. Un-like Pan, he studied the enhancement in soil strength by correlating the model deformation with the relative density.

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Table (1): Required input energy and (n) values for different deposits, after Shi[14]

Source	n values	Energy input (kN·m)	Deposit
Menard and Broise (1975)	1.0	—	All soils
Leonards et al. (1980)	0.5	793	Dry fine to medium sand
Lukas (1980)	0.65–0.8	294–659	Miscellaneous fill
Mayne et al. (1984)	0.3–0.8	4–39,560	Sands and miscellaneous fill
Van Impe and Bouazza (1996)	0.5–0.65	1,290	Municipal solid waste
Kumar and Puri (2001)	0.53	3,570	Sand, gravels, and silt
Miao et al. (2006)	0.57	2,500	Clayed silt and silt
Shui et al. (2006)	0.38	10,000	Backfill and silt
Feng et al. (2010)	0.35	8,000	Backfill, sand, and clay
This study	0.3	6,000	Backfill, silt, and cobble

Xie et al used a 3D finite element model able to deal with large deformation to study the collide phenomenon between tamper and ground; they could predict the ground deformation and stresses values but not the soil enhancement. Generally, most researchers studied the dynamic compaction based on finite element model, starting with simple one-dimension models and up to very sophisticated 3-dimensions models. Although, those models were capable to simulate the ground response to (DC) with reasonable accuracy, but this approach is too complicated to be used commercially. On other hand, the available empirical formulas introduced by Mayne can only predict the depth of influence, peak particle velocity and vertical stresses below tamper. The aim of this research is to introduce more practical approach to predict the ground response to (DC), this approach based on deriving closed-form equations to estimate ground deformation and soil strength enhancement due to (DC) using very basic soil parameters. Zekkos [8] had carried out a survey for more than 50 dynamic compaction Municipal Solid Waste (MSW) sites. Based on Zekkos and others surveys, the practical ranges of (DC) parameters could be summarized as follows:

- Tamper weight:	1-40 ton (Typically 10-20 ton)
- Dropping height:	5-30 m (Typically 10-30 m)
- Depth of improvement	Down to 10-15m (Typically down to 5-10 m)
- Applied energy	30-1000 t.m (Typically 200-600 m)
- Energy per surface area	150-300 t.m/m^2
- Energy per volume	200-1100 KJ/m^3 (0.3-1.8 Standard Proctor Test)
- Number of drops per pass	7-15 drop
- Crater depth	0.7 – 2.0 m
- Soil strength improvement	200-400%

II. DYNAMIC COMPACTION PROCEDURE

The common methodology of (DC) is described in details in FHWA technical report by Lukas [9] and it could be summarized in the following steps:

- Carrying out pre-compaction site investigation to determine the design parameters such as depth, strength, classification and homogeneity of loose layer, in addition to ground water level, any restraints due to near super or underground structures and any other data could affect the compaction plan design.
- Designing compaction plan based on the collected data as follows:
  - Estimating the total required energy per unit volume (specific energy) based on soil classification (cohesive soil needs more energy than granular one) and desire degree of compaction (more compacted soil needs more energy).
  - Selecting tamper weight and dropping height based on classification and depth of loose layer and available equipment, ($D_{max} =$ $n\sqrt{W}$).
  - Determining the grid pattern of compaction based on site plan and tamper diameter (grid spacing should be between 1.5 to 2.5 tamper diameter)
  - Calculating required energy per point by dividing the required specific energy by volume (grid spacing^2 x layer thickness)
  - Calculating the required total number of drops per point by dividing the required energy per point by the energy of single drop (tamper weight x dropping height)
  - Dividing the total number of drops in to several passes (generally, 7-15 drops per point are made in each pass)
- Designed compaction plan may be tested on single point before execution.
- Site should be leveled after each pass and excess water pressure due to (DC) (in case of high ground water level) should be allowed to dissipate before next pass.
- After applying all passes, a surface stabilizing layer (0.3 to 0.9 m) may be applied if needed; it should be compacted by flat tamper (ironing).

- Finally, a post-compaction site investigation should be carried out to ensure the loose layer achieved the required improvement.

III. PROPOSED APPROACH

The proposed approach depends on the following assumptions:

- Collision energy (W.H) will completely transfer to soil

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without any losses

- Collision energy will dissipated in soil with slope 2V:1H (after Bowles[11])
- All deformations in ground due to collision are plastic
- In order to consider the variation is soil parameters, loose soil layer could be divided into set of sub-layers, each one has its own parameters such as elastic and shear modulus, void ratio, N-SPT,..etc.
- Each sub-layer could be modeled by spring with certain stiffness (K), this stiffness equals the axial (or bearing) stiffness of the soil block. Soil block width is calculated based on dissipation slope 2V:1H

$$K = E.A_b/h$$
$$= EB^2/h$$
$$K = E(h+2b)^2/4h \qquad \text{……. (1)}$$

Where:
- B : Soil block width = 0.5 (h + 2b)
- H : Sub-layer thickness
- A_b: Bearing area of soil block = B^2
- b : Tamper width
- E : Elastic modulus of soil

- The behavior of the whole loose soil layer could be simulated be one spring with equivalent stiffness (Keq) where:

$$1/K_{eq} = 1/k_1 + 1/k_2 + \ldots + 1/k_n$$

Where: n : number of sub-layers

Proposed model is shown in figure (2). Applying conservation energy principal,

$$W.H = 0.5\, K_{eq}\Delta^2$$

Where: Δ : the vertical deformation

Hence, $$\Delta = \sqrt{\frac{2W.H}{K_{eq}}} \qquad \text{……. (2)}$$

Equation (2) is a basic formula; it could be applied on the whole loose layer to get its total deformation due to one drop, and it also could be applied on a certain depth starting from ground surface to get the contribution of that depth in total deformation. For example,

Total deformation of loose layer $\Delta_{Total} = \sqrt{\frac{2W.H}{K_{eq\,(1\ to\ n)}}}$

Total deformation of upper 3 sub-layers $\Delta_{(1\text{-}3)} = \sqrt{\frac{2W.H}{K_{eq\,(1\ to\ 3)}}}$

Deformation of 3rd sub-layer $\Delta_3 = \sqrt{\frac{2W.H}{K_{eq\,(1\ to\ 3)}}} - \sqrt{\frac{2W.H}{K_{eq\,(1\ to\ 2)}}}$

For homogenous Loose layer, ($E_1 = E_2 = \ldots\ldots = E_n = E$)

$$K_{eq} = E(D+2b)^2/4D$$
$$\Delta_{Total} = \sqrt{\frac{8W.H.D}{E\,(D+2b)^2}} \qquad \text{……. (3)}$$

Equation (3) is equivalent to equation (2) in homogenous soil where (D) is the thickness of considered surface layer. It could be used to calculate the total deformation of whole layer due to one drop or the contribution of any sub-layer same as equation(2). The accuracy of equation (3) mainly depends on the accuracy of (E) value. Determining the accurate value of soil elastic modulus of each sub-layer experimentally is a long, difficult and expensive process. The other option is to estimate the value of (E) based on the results of common field test such as Standard Penetration Test (SPT) or Cone Penetration Test (CPT).

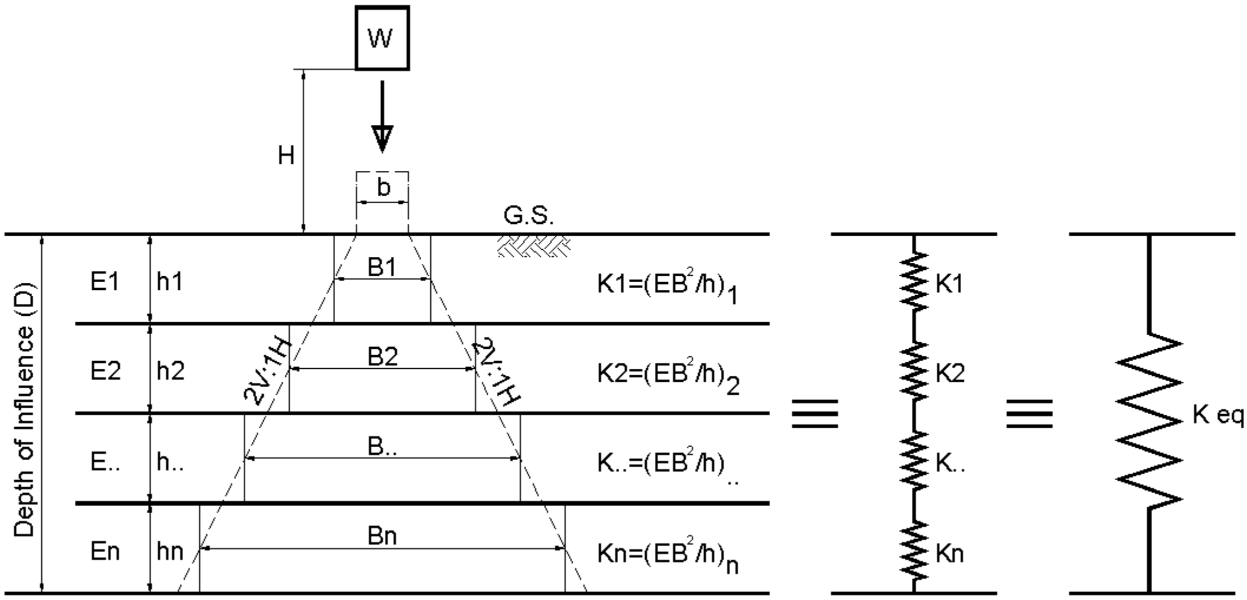

Fig (2): Proposed model

The calculated ground deformations could be used to estimate the enhancement of soil strength by calculating the post-compaction void ratio. (Refer to figure (3))

$$(eo - e) = \frac{Vvo - Vv}{Vs} = \frac{\Delta V}{Vs}$$

$$= \frac{\Delta V}{V} \cdot \frac{V}{Vs} = \frac{\Delta h}{ho}(1+eo)$$

$$e = eo - \frac{\Delta h}{ho}(1+eo) \qquad \text{……. (4)}$$

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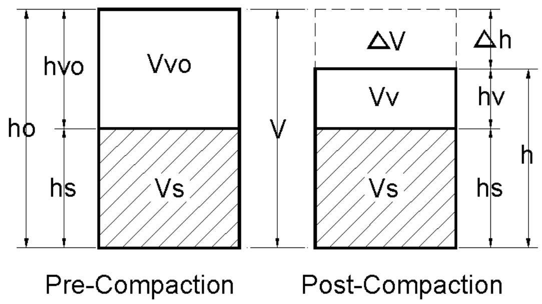

Fig (3): Phase diagram

Void ratio of sand could be correlated directly with field test results such as (SPT) or (CPT), hence; it could be used to update the elastic modulus of the model after each drop. This technique allows simulating the actual non-linear behavior of the soil. Both pre and post compaction soil condition (or strength) determined by site investigation could be presented in terms of different parameters based on the type of applied field tests. Those parameters could be void ratio (e), dry density (γ_{dry}), angle of internal friction (φ), relative density (Dr), or shear wave velocity (Vs). It is not practical to develop different equations for each type of field test, instead, one set of equations is developed to describe soil behavior under dynamic compaction using standard penetration test (SPT) and other parameters could be correlated to (SPT) using empirical formulas. Many geotechnical codes and handbooks presented guide values for soil parameters and empirical formulas to correlate those parameters with common field tests specially (SPT) & (CPT). The most common values obtained from Das [10], Bowles [11] and Day [12] are summarized in table (2), (3). The following empirical formulas correlate soil parameters to (SPT), they are developed using best fitting curve technique on values from table (2):

e $\approx 1.2 - 0.015\,N$ ……. (5)

γ_{dry}(t/m^3) $\approx 1.25 - 0.01\,N$ ……. (6)

φ $\approx 0.35\,N + 27$ ……. (7)

Dr $\approx 0.12\sqrt{N}$ ……. (8)

E (t/m^2) $\approx 3\,N^{2.5}$ ……. (9)

q_c (t/m^2) $\approx 50\,N$ ……. (10)

Vs (m/s) $\approx 110\sqrt[3]{N}$ ……. (11)

Table (2): Empirical relations between shear wave velocity (Vs) and (SPT) test, after Wair [15]

Study	V_s based on N_{60} (m/s)
Ohba & Toriuma (1970)	$82.5\ N_{60}^{0.31}$
Ohsaki & Iwasaki (1973)	$78.0\ N_{60}^{0.39}$
Ohta & Goto (1978)	$82.1\ N_{60}^{0.35}$
Ohta & Goto (1978)	$89.5\ N_{60}^{0.27}$
Ohta & Goto (1978)	$130.3\ N_{60}^{0.27}$
Imai & Tonouchi (1982)	$93.7\ N_{60}^{0.31}$
Imai & Tonouchi (1982)	$105.2\ N_{60}^{0.32}$
Lin et al. (1984)	$62.0\ N_{60}^{0.50}$
Sisman (1995)	$31.0\ N_{60}^{0.51}$
Iyisan (1996)	$48.6\ N_{60}^{0.52}$
Jafari et al. (1997)	$20.0\ N_{60}^{0.85}$
Kiku et al. (2001)	$66.1\ N_{60}^{0.29}$
Hasncebi & Ulusay (2007)	$104.8\ N_{60}^{0.26}$

Total ground deformation could be calculated using (SPT) results by substituting (E) value from equation (9) in equation (3) as shown in equation (12). Similarly, the enhancement in soil strength could be presented in terms of increasing in (SPT) by substituting (e) value from equation (5) in equation (4) as shown in equation (13).

$$\Delta_{Total} = \sqrt{\frac{8\,W.H.D}{3\,N^{2.5}\,(D+2b)^2}} \qquad \text{……. (12)}$$

$$N_{n+1} = N_n + \frac{\Delta_n}{D}(145 - N_n) \qquad \text{……. (13)}$$

Where depth of influence (D) is the minimum of actual loose layer thickness and $(0.3\text{-}0.8)\sqrt{WH}$ after Mayne[3].

Table (3): Summary for guide value of sand parameters

Sand Class.	SPT (N)	e	γ_{dry} (t/m^3)	φ (Degree)	Dr	E (MPa)	CPT, q_c (MPa)	Vs (m/s)
Loose	0-10	1.2-0.9	1.2-1.4	27-32	0.15-0.35	2-30	0-5	150-250
Med.	10-30	0.9-0.7	1.4-1.6	32-36	0.35-0.65	30-200	5-15	250-350
Dense	30-50	0.7-0.5	1.6-1.8	36-40	0.65- 0.85	200-600	15-25	350-450

IV. CASE STUDIES

A. Indianapolis site (Leonards et al. 1980), After Chow [5]

Dynamic compaction (DC) was used to densify a granular fill layer before constructing a warehouse. The fill were loose, fine-to-medium sand. Pre and post compaction site investigation using (CPT) are shown in figure (4-a). Site was compacted by dropping a 5.9 ton pounder from 12m height for 7 times on each point of the grid. Grid spacing was 1.5m and pounder width was 1.22m. As shown in figure (4-a), Initial and final (CPT) test results are 50 and 150 kg/cm^2 respectively. Using equation (10) the equivalent initial and final (SPT) values are 10 and 30 respectively.

Assuming $D = 0.5\sqrt{WH}$, crater depth after 1st drop is calculated using Equations (12), then soil strength updated using equation (13). Then crater depth after 2nd drop is calculated and so on. Relations between measured and calculated crater depth with number of drops are shown in figure (4-b). Table (4) shows calculated crater depth and enhanced (SPT) value after each drop.

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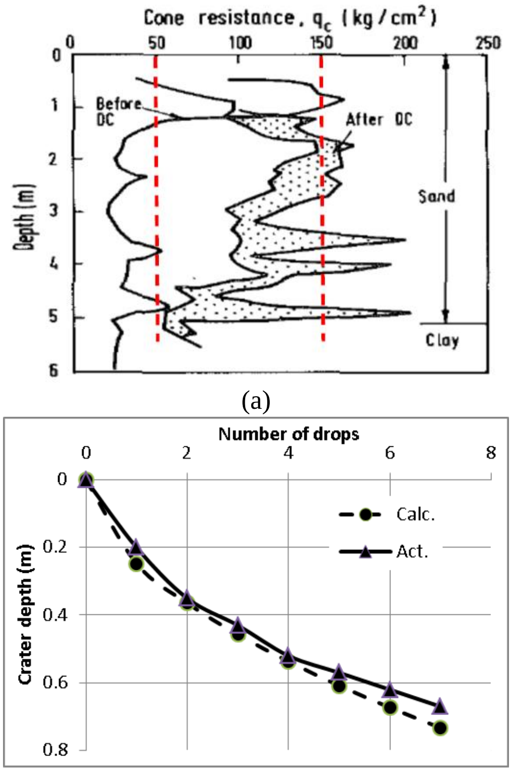


(a)

(b)

Fig (4): Indianapolis site,(a) Pre and post compaction (CPT) values - (b) measured and calculated crater depth

Form table (4), the (SPT) value after the 7th drop is 31 (equivalent to qc = 155 kg/cm^2) which matches the average value in figure (4-a).

Table (4): Indianapolis site: calculated crater depth and (SPT) value after each drop

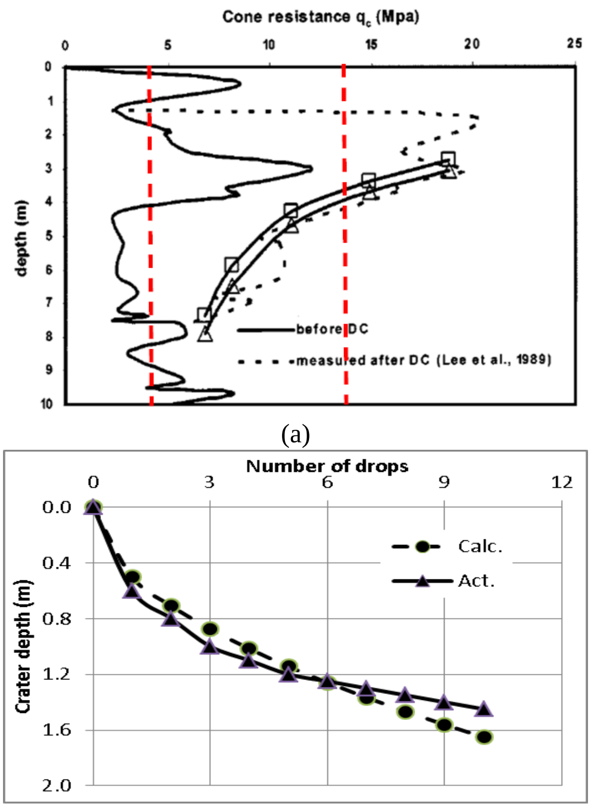
Drop	N	dh	Crater depth	
			Calc.	Act.
1	10.0	0.25	0.25	0.20
2	18.2	0.12	0.36	0.35
3	21.9	0.09	0.46	0.43
4	24.7	0.08	0.54	0.52
5	27.1	0.07	0.61	0.57
6	29.2	0.06	0.67	0.62
7	31.0	0.06	0.73	0.67

B. KampungPakar site, Malaysia (Lee et al. 1989), After Lee[8]

The soil deposit of the site comprises of 14 m of loose sand sandwiching a layer of silty clay between 10 and 12 m depth. Below the lower layer of loose sand is limestone bedrock. The site subjected to dynamic compaction using a 15 t and 1.8 mx1.8 m square tamper were carried out over a 6 mx6 m grid pattern. In the first pass, the tamper was dropped 10 times from 20 m. Figure (5-a), (5-b) shows the pre and post compaction (CPT) values, measured and calculated crater depth respectively. As shown in Figure (5-a), Initial and final (CPT) test results are 4 and 14 MPa respectively which equivalent to (SPT) initial and final (SPT) values are 8 and 30 respectively. Table (5) summarized the calculated crater depth and enhanced (SPT) value after each drop. It shows that final (SPT) value is 32 (equivalent to qc = 16 MPa) which is very close to the average value in figure (5-a).

Table (5): Kampung Pakarsite: calculated crater depth and (SPT) value after each drop

Drop	N	dh	Crater depth	
			Calc.	Act.
1	8.0	0.50	0.50	0.60
2	16.2	0.21	0.71	0.80
3	19.4	0.17	0.88	1.00
4	21.9	0.14	1.02	1.10
5	24.0	0.13	1.14	1.20
6	25.9	0.12	1.26	1.25
7	27.5	0.11	1.37	1.30
8	29.1	0.10	1.47	1.35
9	30.5	0.09	1.56	1.40
10	31.8	0.09	1.65	1.45



(a)

(b)

Fig (5): KampungPakarsite,(a) Pre and post compaction (CPT) values - (b) measured and calculated crater depth

C. Daya Bay, Guangdong Province, China. (Shi et al. 2011) [13]

The site covers a beach area adjacent to the South China Sea. The typical soil profile consisted of three distinct layers

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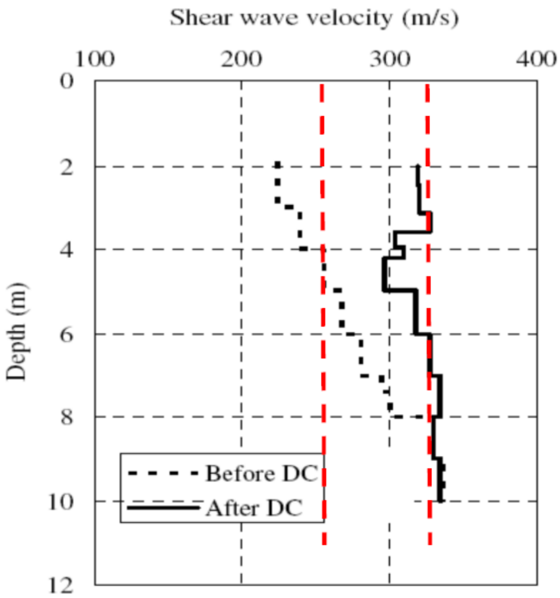
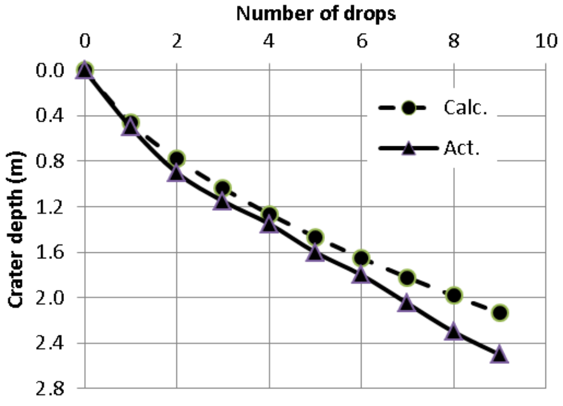
with poor homogeneity, resting on partly weathered bedrock. The uppermost soil layer was approximately 4.6 m thick and it was composed of recently backfilled, highly weathered loose gravels and sandstones with traces of clay, silt, and weathered rock fragments. Then, a silt layer of approximately 2.2 m in thicknesswas found, which also contained some gravel with poor cohesion. Finally, a cobble layer was encountered. This layer featuredcobbles of poor particle gradation. Pre and post compaction soil strength was evaluated based on spectral analysis of surface wave (SASW) test. As shown in figure (6-a), the average initial and final shear wave velocities are 250 and 320 m/s respectively which equivalent to (SPT) values of 12 and 27 respectively. Figure (6-b) shows variation of both calculated and measured crater depth with number of drops, the graph shows acceptable matching between the two curves. Summarized calculation results listed in table (6) shows that final calculated (SPT) is 29 (equivalent to Vs = 335 m/s) while the measured final Vs is 320 m/s

Table (6): Daya Bay site: calculated crater depth and (SPT) value after each drop

Drop	N	dh	Crater depth	
			Calc.	Act.
1	12.0	0.46	0.46	0.50
2	16.3	0.32	0.78	0.90
3	19.1	0.26	1.04	1.15
4	21.3	0.23	1.26	1.35
5	23.3	0.20	1.47	1.60
6	25.0	0.18	1.65	1.80
7	26.5	0.17	1.82	2.05
8	27.9	0.16	1.98	2.30
9	29.2	0.15	2.13	2.50



(a)

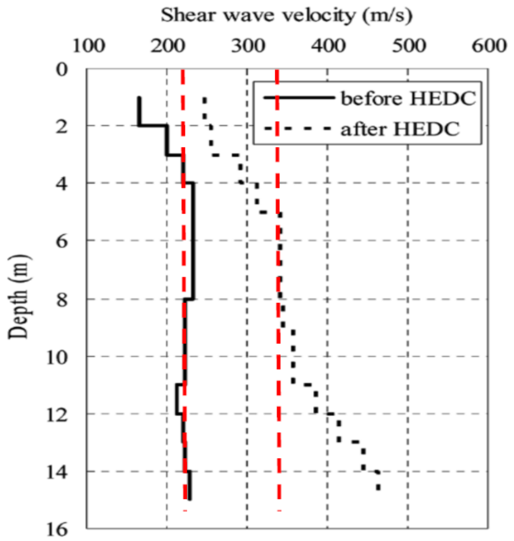


(b)

Fig (6): Daya Bay site,(a) Pre and post compaction (Vs) values - (b) measured and calculated crater depth

D. Mabianzhou Island site, China (Shi et al. 2010) [14]

Project site is located on Mabianzhou Island, in Huizhou City, Guangdong Province, China. The project was constructed on the backfilled soil ground in the coastal reclamation area. The backfilled soil includes massive coarse-grained gravels. Dynamic compaction program was dropping 2m diameter tamper weights 40 ton from 30 m height 15 times on each point. Soil profile consists of 9.0m thick loose fill layer laid on lower 7.5 m thick silty clay layer. Pre and post compaction soil strength was evaluated based on spectral analysis of surface wave (SASW) test. As shown in figure (7-a), the average initial and final shear wave velocities are 220 and 340 m/s respectively which equivalent to (SPT) values of 8 and 30 respectively. Figure (7-b) shows variation of both calculated and measured crater depth with number of drops, the graph shows acceptable matching between the two curves. Summarized calculation results listed in table (7) shows that final calculated (SPT) is 37 (equivalent to Vs = 365 m/s) while the measured final Vs is 340 m/s



(a)

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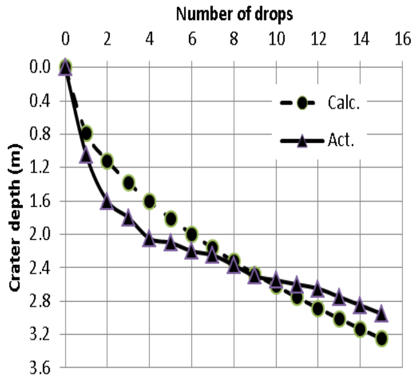

(b)

Fig (7): Mabianzhou Island site,(a) Pre and post compaction (Vs) values - (b) measured and calculated crater depth

Table (7): Mabianzhou Island site: calculated crater depth and (SPT) value after each drop.

Drop	N	dh	Crater depth	
			Calc.	Act.
1	8.0	0.79	0.79	1.05
2	16.0	0.33	1.12	1.60
3	19.1	0.26	1.38	1.80
4	21.6	0.23	1.61	2.05
5	23.7	0.20	1.81	2.10
6	25.5	0.18	2.00	2.20
7	27.2	0.17	2.17	2.25
8	28.7	0.16	2.33	2.37
9	30.0	0.15	2.48	2.50
10	31.3	0.14	2.62	2.55
11	32.5	0.14	2.76	2.60
12	33.7	0.13	2.89	2.65
13	34.8	0.13	3.01	2.75
14	35.8	0.12	3.13	2.85
15	36.8	0.12	3.25	2.95

V. CONCLUSIONS

The results of previous study could be concluded in the following points:

- The proposed approach consists of two equations, the 1stone used to calculate the ground settlement due to one temper drop, while the 2ndone used to calculate the updated soil parameters due to the ground settlement from the previous drop
- By applying the two equations successively, both ground settlement and soil parameters improvement could be calculated after each tamper drop.
- The proposed approach was applied on four case studies and its results were so close to measured ones.
- The proposed approach could be used in designing or testing the dynamic compaction procedures and also in monitoring the quality of execution by comparing the measured settlement after each drop with calculated one.
- Although, the proposed approach is using (SPT) values to calculate the ground settlement and update its strength, but it could be applied using other field test by converting their results to equivalent (SPT)

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AUTHOR'S PROFILE

Ahmed M. Ebid. He received his Ph.D. in soil mechanics from Ain-Shams Uni.-Egypt 2004.He is a consultant in designing concrete structures since 2012, He published 7researches is geotechnical engineering, soil mechanics, repairing using FRP and optimizing the design of concrete elements. Presently, he isa lecturein structural department, faculty of engineering, Future Uni.-Egypt.