Research Article

Deformation Analysis of Granular Soils under Dynamic Compaction Based on Stochastic Medium Theory

Jifang Du ¹, Shuaifeng Wu ², Sen Hou,³ and Yingqi Wei²

¹Department of Transportation Science and Engineering, Beihang University, Beijing 100191, China

²State Key Laboratory of Simulation and Regulation of Water Cycle in River Basin, China Institute of Water Resources and Hydropower Research, Beijing 100048, China

³The Engineering Research Base of China Airport Construction Corporation of CAAC, Beijing 100101, China

Correspondence should be addressed to Shuaifeng Wu; wusf@iwhr.com

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Dynamic compaction (DC) is widely used to improve the mechanical properties of soils and other granular fill material upon which foundations or other structures are to be built. To calculate the inner deformation induced by DC, a computational model based on stochastic medium theory was developed to deduce the amount of deformation in the fill from the geometry of the DC crater. For this model, the tamper-soil system was simplified to an axisymmetric geometry and the probability of any unit of material below the crater being deformed can be calculated. Subsequently, the deformation at any point can be obtained by integration. Input parameters for the model were established by back analysis using the results from a DC field test and published data. Comparing the model results to results from actual and simulated DC programs shows that the computational model is useful for calculating deformation induced by DC without the need to consider the constitutive model of the soil.

1. Introduction

Compacting the soil that will be under a foundation is an essential procedure to ensure the stability of the structure. Improving the soil beneath the foundation can be accomplished by a number of different methods including static or dynamic rollers and fiber reinforcement [1–3]. However, static or dynamic rollers are limited and can only compact the soil to a relatively shallow depth (0.2–1.0 m) [4] and fiber reinforcement is expensive and not environment-friendly for large construction projects [5]. Compared with the above methods, dynamic compaction (DC) is a suitable method with the following advantages: (1) high density compaction to a considerable depth; (2) wide applicability in soils with different soil properties and different water contents; (3) simplicity of operation using widely available and simple equipment; (4) low cost. Menard [6] first introduced DC for construction and conducted systematically research on the method. Dynamic compaction simply involves dropping a

tamper (10–40 t) from a height of 10–20 m onto the ground. Several blows are repeated until the ground under the tamper reaches the desired degree of soil improvement.

The extent of soil improvement is one of the most important concerns for a DC project; it is the essential indicator of the project's success. The soil improvement can be divided into two parts: the depth of improvement (*DI*) and the lateral extent of improvement (*LI*).

The *DI* is a major issue facing the DC program designers and contractors [7]. The first equation for predicting *DI* was proposed by Menard [6] and it can be expressed as $DI = \sqrt{MH}$, where *M* is the tamper weight in tones and *H* is the falling height in meters. Mayne [8] summarized 120 sites with diverse soil properties and modified this equation to the form:

$$DI = n\sqrt{MH} \quad (1)$$

where *n* (commonly equal to 0.3–0.8) is an empirical constant related to soil properties. Subsequent researches by Leonards

[9], Lukas [10], Rollins [11], Slocombe [12], and Luoguo [13] calibrated and determined different values for n and proposed other forms for the equation based on the energy applied. However, these equations can only roughly evaluate the DI and do not consider factors like soil properties and tamper radius. Although researchers like Chow [14] and Smits [15] proposed computational models that did consider these factors, the parameters in their models are difficult to measure, and the values these factors have affect the results.

The LI is the maximum width the improvement extends horizontally away from the tamper. It is an essential parameter for determining the optimum distance between tamping points. There has been little research on how to calculate LI . Poran [16] discussed the factors that influence the degree of improvement and divided these factors into site-dependent factors and equipment-dependent factors. Two equations, one for the input energy and one for the dimensions of the plastic zone, were derived to evaluate the depth and width of improvement. The plastic zone was assumed to be semispherical shape.

In recent years, the dynamic mechanisms of DC have undergone further analysis and many simulations have been developed using finite or discrete element methods. For instance, Lee and Gu [17] proposed a method for estimating the degree and depth of improvement based on a two-dimensional finite element model. Ma [4] used the PFC2D/PFC3D software (Itasca Consulting Group, Inc., Minneapolis, MN, USA) to determine improvement and DI by analyzing porosity variations. Mostafa [18] conducted a comprehensive parametric study on the behavior of cohesive soils. Jia [19] investigated the behavior of granular soils during DC using PFC/FLAC software (Itasca Consulting Group). However, preparing these computational models and simulations is very complicated and time-consuming and it is not really feasible to use them in the field at construction sites.

During DC, the deformation can reflect the degree of soil improvement; this is a good indicator of how much the soil has been compacted. In most cases, when the soil in the crater is depressed less than a specified amounts per drop for the final two drops, it can be assumed that the soil's strength has been improved enough to meet the project's design requirements. The deformation in the soil can also be a reflection of the change in soil properties and used to determine how much the soil has been reinforced by the DC.

Stochastic medium theory was initially proposed by Litwiniszyn [20] and developed further by Liu et al. [21, 22]. The stochastic medium in question is granular material like sand, gravel, and rock fragments. The excavation of underground space often causes deformation of the overlying strata [23, 24] and the theory is mainly used to calculate subsidence after an opening has been excavated. The subsidence caused by the excavation will be transferred to the overlying strata (Figure 1(a)). For DC, the settling is caused by the impact of the tamper. Figure 1(b) shows soil particles compressed by tamping. Comparing Figure 1(a) with Figure 1(b) shows that the two kinds of settling are similar for noncohesive soils.

In this paper, the deformation caused by the DC is modeled using stochastic medium theory. A computational

model is proposed based on this theory and two parameters, a compression factor (determined by the type of DC equipment used) and an influence angle (a function of the soil's properties). Finally, the applicability of this model is examined by comparing results calculated using the model with measurement made after a field test and with published data. This study can provide a method for determining how well a DC program is reinforcing the material upon which a foundation will be installed and will be valuable for the design and construction of DC projects.

2. Model for Dynamic Compaction

2.1. Introduction to Stochastic Medium Theory. In an Eulerian coordinate system, assume that the probability density for deformation of dx is $f(x^2)$ for the z -axis symmetry of the deformation curve. The deformation probability P for dx can be expressed as

$$P(dx) = f(x^2) dx. \quad (2)$$

For an infinitesimal plane $ds = dxdy$ centered at point $A(x, y, z)$, as shown in Figure 2(a), the deformation probability is

$$\begin{aligned} P(ds) &= P(dx)P(dy) = f(x^2)dx f(y^2)dy \\ &= f(x^2)f(y^2)ds \end{aligned} \quad (3)$$

If the coordinate system (x, y, z) is changed to (x_1, y_1, z) as shown in Figure 2(b), then (3) is transformed into

$$\begin{aligned} P(ds) &= P(dx_1)P(dy_1) = f(x_1^2)dx_1 f(y_1^2)dy_1 \\ &= f(x_1^2)f(y_1^2)ds. \end{aligned} \quad (4)$$

If the x_1 axis passes through the point A , then

$$\begin{aligned} x_1^2 &= x^2 + y^2, \\ y_1 &= 0 \end{aligned} \quad (5)$$

$$\begin{aligned} f(x_1^2)f(y_1^2)ds &= f(x^2 + y^2)f(0)ds \\ &= cf(x^2 + y^2)ds \end{aligned} \quad (6)$$

$$f(x^2) \cdot f(y^2) = f(x_1^2)f(y_1^2) = c \cdot f(x^2 + y^2) \quad (7)$$

in which c is a constant.

The derivation of (7) will yield:

$$\begin{aligned} f(y^2) \frac{\partial f(x^2)}{\partial x^2} &= c \frac{\partial f(x^2 + y^2)}{\partial (x^2 + y^2)} \frac{\partial (x^2 + y^2)}{\partial x^2} \\ &= c \frac{\partial f(x^2 + y^2)}{\partial (x^2 + y^2)} \\ f(x^2) \frac{\partial f(y^2)}{\partial y^2} &= c \frac{\partial f(x^2 + y^2)}{\partial (x^2 + y^2)} \frac{\partial (x^2 + y^2)}{\partial y^2} \\ &= c \frac{\partial f(x^2 + y^2)}{\partial (x^2 + y^2)}; \end{aligned} \quad (8)$$

FIGURE 1: Sketches showing the deformation induced by (a) mining subsidence and (b) dynamic compaction.

FIGURE 2: Diagrams showing the deformation of point A in a coordinate system and in a transformed coordinate system for the DC calculation model.

therefore

$$f(y^2) \frac{\partial f(x^2)}{\partial x^2} = f(x^2) \frac{\partial f(y^2)}{\partial y^2} \quad (9)$$

$$\text{or } \frac{1}{f(x^2)} \frac{\partial f(x^2)}{\partial x^2} = \frac{1}{f(y^2)} \frac{\partial f(y^2)}{\partial y^2}.$$

Equation (9) shows that the two sides of the equation are independent of x and y .

Let

$$\frac{1}{f(x^2)} \frac{\partial f(x^2)}{\partial x^2} = k_1^2. \quad (10)$$

Then, after integration, the following equation can be obtained:

$$f(x^2) = k_2 e^{-k_1^2 x^2} \quad (11)$$

where k_1 and k_2 are parameters corresponding to the depth, z .

Therefore, the deformation probability of plane ds is

$$P(ds) = f(x^2) f(y^2) dx dy = k_2^2 e^{-k_1^2 (x^2 + y^2)} dx dy. \quad (12)$$

Assuming that the soil is not compacted, the deformation should be equal to the volume of the unit (V_e) in the crater, so

$$V_e = \int_{-\infty}^{+\infty} \int_{-\infty}^{+\infty} f(x^2) f(y^2) dx dy = 1. \quad (13)$$

After solving, it can be seen that

$$k_2^2 = \frac{\pi}{k_1^2}. \quad (14)$$

Based on stochastic medium theory, the parameter k_1 can be expressed as $r(z)$ (also called the influence radius) and $r(z)$ is defined as

$$r(z) = \frac{z}{\tan \beta} \quad (15)$$

where z is the depth of ds and β is the influence angle.

Therefore, the deformation at the point (x, y, z) induced by a unit in the crater can be expressed as

$$W_e = \frac{1}{r^2(z)} \exp \frac{-\pi(x^2 + y^2)}{r^2(z)}. \quad (16)$$

2.2. Computational Model. Assuming that the foundation surface is smooth and the face of the tamper is circular, the crater formed by the tamper can be simplified as a frustum of a cone (Figure 3) and the DC can be simplified into an axisymmetric problem. To simplify the calculation, we only study the deformation in plane zox of the coordinate system. The deformation of point A induced by the unit in the crater can be divided into two directions: the vertical and the horizontal, which are W_e and U_e in Figure 2(a), respectively. The vertical deformation is much easier to be detected than the horizontal. Therefore, in this paper, only vertical displacement is considered; horizontal displacement is ignored. Taking the central axis of the crater as the z -axis, for a unit $ds = dx_e dy_e dz_e$ centered at (x_e, y_e, z_e) in the crater, the induced deformation at point $A(x, y, z)$ (Figure 2) can be expressed by

$$W_e = \frac{1}{r^2 (z - z_e)} \cdot \exp \frac{-\pi [(x - x_e)^2 + (y - y_e)^2]}{r^2 (z - z_e)} dx_e dy_e dz_e. \quad (17)$$

The deformation induced in the crater can be obtained by integration, so

$$W(x, y, z) = \iiint_{\Omega} \frac{1}{r^2 (z - z_e)} \exp \frac{-\pi [(x - x_e)^2 + (y - y_e)^2]}{r^2 (z - z_e)} dx_e dy_e dz_e \quad (18)$$

where Ω represents the whole region of the crater.

The crater's coordinate system can be transformed into cylindrical coordinates and the unit in the crater changed from $dx_e dy_e dz_e$ to $\rho d\theta d\rho d\zeta$. The geometric parameters of the crater (Figure 3) are the depth, h , and the depth within the $(0, h)$ range represented by ζ ; a vertical section through the crater is trapezoidal and the radius at different crater depths can be expressed as $\rho(\zeta)$ which can be calculated from

$$\rho(\zeta) = \rho_1 + \frac{(\rho_2 - \rho_1)\zeta}{h} \quad (19)$$

where ρ_1 and ρ_2 are the crater's top and the bottom radii.

The deformation at any depth z can be calculated from

$$\begin{aligned} W(x, z) &= \int_0^h \int_0^{\rho(\zeta)} \int_0^{2\pi} \frac{1}{r^2 (z - \zeta)} \exp \left(-\pi \frac{(x - \rho \cos \theta)^2 + (0 - \rho \sin \theta)^2}{r^2 (z - \zeta)} \right) \\ &\quad \cdot \rho d\theta d\rho d\zeta \\ &= \eta \int_0^h \int_0^{\rho(\zeta)} \int_0^{2\pi} \frac{\tan^2 \beta}{(z - \zeta)^2} \exp \left(-\pi \tan^2 \beta \frac{(x - \rho \cos \theta)^2 + (\rho \sin \theta)^2}{(z - \zeta)^2} \right) \\ &\quad \cdot \rho d\theta d\rho d\zeta \end{aligned} \quad (20)$$

Dynamic compaction makes the soil denser. Therefore, a compression coefficient, η , must be introduced to calculate the vertical movement of a unit of compacted material.

The influence angle β can reflect the properties of the soil at depth z . However, if the soil at the construction site

is considered to be homogeneous, the influence angle will be constant. For DC, the impact energy influence angle is the main factor affecting the amount of compaction. Therefore, the impact energy influence angle β is used in the calculations in this paper. According to Yao [25], the DC impact influence angle for gravelly soil can be assumed to be 30° from the horizontal, so 30° is used as the influence angle in all the calculations presented below. The subsidence at a specific depth can be calculated using (20); the only unknown is η , the compression coefficient.

3. Model Verification

3.1. Field Test Results. To acquire values for the variables in the model, a field test was conducted at the Chengde airport construction site, Hebei Province, China. The construction material was mainly crushed rock that complied with airport fill technical code MH/T5035-2017 [26]. Particle gradation curves for the eight different 1-m layers in the fill are shown in Figure 4.

The machine used for the DC was a 70-t crawler. The tamper weighed 16.8 t and was 2.5 m in diameter. The tamper imparted a DC energy of 1000 kNm with a fall of 6 m. To show the amount of settling in the different fill layers caused by the DC, thin plastic sheeting was laid down at three different horizons as the fill was being placed. The sheeting was installed at depths of 1.85 m, 2.8 m, and 4 m below the final fill surface as shown in Figures 5 and 6. For the calculations below, the geometry of the crater was simplified to make the crater a cylinder with a radius of 1.2 m and the parameters used in the calculations were $\rho_1 = \rho_2 = 1.2$ m, $h = 0.8$ m.

The most appropriate value of η is the lowest least squares value for η calculated using

$$R(\eta) = \sum (W_i - W_i^*)^2, \quad (21)$$

where W_i and W_i^* are the calculated and observed results for deformation, respectively. The lower the $R(\eta)$, the better the match between the measured and the calculated result.

The observed and the calculated data are listed in Table 1. The calculated values are obviously larger than the observed values because the calculated values do not take the compression coefficient into account. The relationship between $R(\eta)$ and η was calculated using (21) and the results are shown in Figure 7. As shown in the figure, $R(\eta)$ is minimized when $\eta = 0.65$. The modified data listed in Table 1 were calculated with $\eta = 0.65$. The final deformation is shown in Figure 8; the modified values agreed well with the observed values.

3.2. Published Data (Ghassemi et al., [27]). The below-surface deformation induced by DC is rarely reported in field test results. Therefore, the results from this study are compared with the results from a DC simulation published by Ghassemi et al. [27]. Ghassemi et al. used finite element software to simulate sand compaction and verified the results of Oshima and Takada's [28] centrifuge tests. The simulated DC energy was 4000 kNm, energy equivalent to a 20-t tamper dropping 20 m. Figure 1 in [27] shows the deformation after DC. The

TABLE 1: Calculated, observed, and modified deformation for the fill at the three horizons covered by plastic sheeting at the field test site at different horizontal distances from the tamper (unit: cm).

Depth		Horizontal distance					
		0 m	1 m	2 m	3 m	4 m	5 m
1.85 m	Calculated	42	29.27	9.62	1.59	0.156	0.001
	Observed	25.5	20.5	10	5	0	0
	Modified	27.3	19.03	6.25	1.03	0.1	0
2.8 m	Calculated	18.8	15.9	9.69	4.29	1.403	0.349
	Observed	15.5	15.5	8	3	1	0
	Modified	12.22	10.34	6.3	2.79	0.9	0.23
4 m	Calculated	8.88	8.215	6.502	4.41	2.57	1.29
	Observed	6	6	4	2.5	1.5	0
	Modified	5.8	5.4	4.2	2.9	1.67	0.84

Note: Modified value = Calculated value $\times$ Compression Coefficient

FIGURE 3: Cross-section showing the idealized geometry of the crater created by a circular tamper during dynamic compaction.

FIGURE 4: Particle gradation curves for the 1-m layers in the fill at the Chengde airport construction site used for the field DC test.

dimensions of the simulated crater in Ghassemi et al. were $\rho_1 = 1.4$ m, $\rho_2 = 1.2$ m, $h = 1.2$ m. Those crater dimensions were entered into (20) to calculate the soil deformation for that tamper configuration; the results are listed in Table 2.

FIGURE 5: Sketch showing the layout of the field test and the positions of the plastic sheeting used to monitor the amount of fill settling.

FIGURE 6: A worker laying out plastic sheeting on the top of a 1-m fill layer that will be buried by subsequent layers of fill.

Figure 9 shows that η for this DC simulated configuration is 0.7, so that value was used to calculate the “Modified” results listed in Table 2. As shown in Figure 10, the modified results show a good correlation with the data from Ghassemi et al.

FIGURE 7: Graph showing the relationship between the compression factor, η , and the least square variance between observed and calculated results, $R(\eta)$, for the field test data.

FIGURE 9: Graph showing the relationship between the compression factor, η , and the least square variance between observed and calculated results, $R(\eta)$, for the data published by Ghassemi et al. [27].

FIGURE 8: Cross-section showing the upper 4 m of the fill (the center of the DC crater is at horizontal position “0”) and the observed and modified results at different depth.

FIGURE 10: Graph comparing the deformation values from Ghassemi et al.'s [27] simulated DC on sand to deformation values calculated from the dimensions of Ghassemi's crater using the calculation model developed for this study.

4. Discussion

4.1. Critical Deformation. The computational model has shown its applicability for DC analysis. However, the primary shortcoming of this model is that the calculated deformation cannot be zero even for locations far removed from the tamped crater. This means that the zero deformation boundary cannot be determined using this model. To compensate for this shortcoming, the concept of a critical level of deformation has been formulated. The critical level of deformation is defined as deformation sufficient to significantly reinforce

the soil. Deformation below this level will just modify the soil but not increase its strength. By comparing the Standard penetration test N_{120} (SPT) values from the fill at the Chengde airport construction site before and after DC (Figure 11), the rate of improvement can be calculated. Equation (22) can be used to quantify the amount of improvement.

$$\omega = \frac{SPT \text{ after DC} - SPT \text{ before DC}}{SPT \text{ before DC}} \times 100\% \quad (22)$$

where ω is the rate of improvement.

TABLE 2: Calculated, observed, and modified deformations for simulated sand fill. Observed data are from Ghassemi et al. [27]; calculated and modified data are from the computational model applied to Ghassemi's data (unit: cm).

Depth		Horizontal distance								
		0 m	0.4 m	0.8 m	1.2 m	1.6 m	2 m	2.4 m	2.8 m	3.2 m
1.6 m	Calculated	94.1	89.7	75	49.3	24.1	10.2	4	1.58	0.48
	Observed	64.6	64	55.1	40.8	18.9	7.7	0	0	0
	Modified	65.8	62.8	52.5	34.5	16.9	7.14	2.8	1.1	0.33
2.4 m	Calculated	50.3	48.1	42	33.6	24.6	16.6	10.4	6.1	2.6
	Observed	42.5	37	33.4	25.4	18.3	13.9	8.6	5.3	4.6
	Modified	35.2	33.6	29.4	23.5	17.3	11.6	7.27	4.26	1.84
3.2 m	Calculated	26.8	26.2	24.4	21.6	18.3	14.8	11.5	8.5	4.55
	Observed	20.3	19.7	16.1	11.8	5.9	5.39	3	0	0
	Modified	18.8	18.3	17.1	15.1	12.8	10.4	8	6	3.2
4 m	Calculated	16.26	16	15.35	14.3	12.9	11.4	9.7	8	4.7
	Observed	8.9	7.6	7.6	4.8	4.4	0	0	0	0
	Modified	11.4	11.2	10.7	10	9.06	8	6.8	5.6	3.3
4.8 m	Calculated	10.8	10.7	10.4	9.94	9.3	8.55	7.7	6.8	4.1
	Observed	7	6	6	0	0	0	0	0	0
	Modified	7.56	7.5	7.3	6.9	6.5	6	5.4	4.8	2.9

Note: Modified value = Calculated value $\times$ Compression Coefficient

FIGURE 11: Dynamic cone penetration test results (N_{120}) from the fill at the Chengde airport construction site before and after DC.

Figure 12 is a graph showing the rate of improvement for a number of points in the fill plotted against the corresponding inner deformation. The fitted equation for the curve on that figure is

$$W = 0.28e^{\omega/0.65} + 3.5. \quad (23)$$

According to SPT's performed at the construction site, the DI where the rate of improvement is 50% is about 5 m. The vertical deformation of points at the 5-m depth, from (22), is 0.04 m. Therefore, 0.04 m was determined to be the amount

FIGURE 12: Graph showing the percentage of SPT penetration improvement for several points in the Chengde airport construction site fill after DC.

of deformation that defines the critical level of deformation boundary. Points within that boundary should have had their strength improved by at least 50%. The boundary's location is defined by points with a vertical deformation of 0.04 m; a line drawn through those points is shown in Figure 13. The line shows that the reinforced area is bubble-shaped and the reinforced area extends nearly 3 m from the center of the crater. The area of reinforcement is consisted with the field test results.

4.2. Crater Depth and the Reinforcement Extent. The DI and LI away from the tamping sites are two essential parameters that must be considered when a DC program is being

FIGURE 13: Cross-section showing the “critical level of deformation” boundary, the line within which deformation exceeds 0.04 m.

FIGURE 14: The change of inner deformation with different crater depth along the (a) vertical and (b) horizontal direction.

designed. These parameters are commonly determined by using the Menard equation or practical field experience; both are largely based on the energy produced by the DC. However, for soils with different properties, the same impact energy produces different degrees of reinforcement. Because it is difficult to observe the change that have taken place in the soil after DC, the tamper-soil system is somewhat of a black box in that we only know the inputs to the system but cannot easily monitor the results. However, an “external interface” for this “black box” exists on the geometry of the crater. For the same DC energy, the shapes and the depths of craters are different for different soils. The crater is a reflection of the soil-tamper system and can be used to analyze the

posttamping changes in the soil. In our stochastic-medium-based computational model, the crater is used as the main indicator of soil reinforcement. Figure 14 is the deformation of different depth with different crater depth calculated by the model. The deformation is decreased with the increase of the depth and radial distance. Meanwhile, the increase of the crater depth is corresponding to a larger reinforcement extent.

Figures 15(a) and 15(b) present the relationships between DI and crater depth, LI and crater depth, respectively. The DI increases with the growth of the crater depth and the correlation is very nearly linear. The LI shows the same character with the DI , but the trend of growth slows down

FIGURE 15: The relationship of crater depth and (a) DI , depth of soil improvement, and (b) LI , lateral extent of soil improvement.

with the increase of crater depth. When the crater depth increases from 1 m to 2 m, the growth of LI is about 1.2 m. However, the growth of LI is just 0.6 m when the crater depth increases from 2 m to 3 m. Therefore, it can be concluded that, increasing the crater depth, the DI can be significantly expanded while LI increases consistently with a slowly decreasing rate (Figure 15(b)).

5. Conclusions

A computational model based on stochastic medium theory has been developed to determine how much dynamic compaction (DC) reinforces the soil. This model is verified by comparing model results with the results from two compaction programs, one on crushed rock fill and the other simulating a sand fill. The compression coefficients for these two programs are 0.65 for the crushed rock and 0.7 for the sand. This method for establishing the degree of reinforcement has several advantages when compared with traditional methods. First, it takes the geometry of the crater into account. The shape and depth of the crater can reflect the DC energies and the soil properties and the crater's dimension are easy to obtain at the construction site with no special equipment or personnel training required. Second, the model assesses the extent of the reinforcement both laterally and vertically instead of merely estimating the DI . Finally, the computation is simple and can be easily solved using commercially available programs. However, this model is a prototype and more details should be studied. Additional validation of the computational model is also needed before sufficient confidence on the model's results can be established.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare no conflicts of interest.

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