The Effect of Distance between Sheet Pile and Foundation on Bearing Capacity of Foundation

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1. Introduction

The increase in urban populations has led to an increase in the need for transportation systems and residential areas. Despite this increase, the useable areas in cities are limited. Therefore, it has become necessary to use existing areas more effectively. New living quarters are often created by deep excavations or by rehabilitating dangerous dip slopes. This has brought stability and safety problems around dangerous dip slopes and deep excavations. During the construction of foundations or deep excavations in urban areas, slope slips and large settlements may occur [1–7]. Lateral support systems are used to prevent dangerous deformations around such construction sites. One of the common lateral support systems is SPWs. SPWs are flexible and generally waterproof structures. They are also used to limit horizontal displacements of the soil mass as lateral support. The reasons SPWs are preferred are that they are economical and save time and space.

Safety and economic efficiency are key objectives in the engineering design [8]. In deep excavations, SPWs are designed according to wall displacement to evaluate their safety and economic efficiency. In this evaluation, two extreme conditions are taken into consideration as follows: one of them is very small deformations, which means that SPW is designed uneconomically; the other one is large displacements, which cause safety problems around the excavation area during construction.

As a result of literature research, some of the studies on excavation stability and safety are as follows: in some papers, numerical studies have been carried out using data from case analyses. In other research studies in the literature are investigated excavation width, wall displacements in the corners of the excavations, support geometry on wall and soil movements, the soil and wall interface, and the effects of wall elasticity are effective on wall-soil deformations. Results from these studies was mostly compared with case studies [9–14].

In some papers, only numerical analyses were carried out using typical values of soil strength parameters. These studies modeled the construction phase of the diaphragm wall using 3D FEM, generally investigated soil stress distribution mechanisms and soil displacements, stability parameters of 3D rectangular, elastoplastic evaluation of the soil and retaining structure, and wall thickness and wall...
penetration depth on retaining walls in their analysis [15–24].

Apart from numerical studies, there are also studies similar to this study. For example, Tan et al. [23] examined the systems for the settlement of buildings adjoining excavations. As a result of their study, it was determined that both the buildings on shallow foundations and those on short piles were sensitive to damage caused by adjoining to deep excavation. This result is different from the popular opinion in the literature.

In the literature, it was seen that the behavior of retaining walls on cohesive soils was examined in general. Numerous studies have been conducted on the behavior that may occur as a result of excavations on retaining walls built on cohesive soils [25–33].

There has been a limited number of literature on the behavior of retaining walls built on sand. These studies were conducted on sand in which an internal friction an angle below 32° was used and the deformations of SPW were conducted on sand in which an internal friction angle than other sands.

1.1. Research Significance. There are several parameters that affect the bearing capacity of the foundations (qult) near the retaining structure and the deformations of the retaining structure. These parameters are relative density (Df), SPW length (Hf), excavation width, the distance between foundation and retaining structure (L), and penetration depth (Hp). It was observed that the effect of only one of these parameters (generally Df or Hp) on qult was examined in the literature. The effect of the L parameter on qult and SPW deformations (δmax) has not been encountered in previous studies. Besides, as can be seen from the literature summary above, some studies were based only on numerical analysis. In some of them, numerical analyses were made using the measurements in the field. This study was based on the effects of L and Hp parameters on qult and δmax. Except for these two parameters, all other parameters were kept constant.

The soils in the field are generally not homogeneous and consist of different soil layers. The difficulty of determining the soil strength parameters of all layers and the unknown interaction behavior between the layers complicates the stability problems. In such soils, strength parameters related to the soil layers, which are generally accepted strength parameters or obtained from drilling data, are used. This study was based on laboratory experiments and numerical analysis. By simulating a homogeneous soil profile in the laboratory, multilayer effects were prevented and model experiments were carried out. The results obtained were verified by numerical analysis.

2. Material Properties of the Sand and Sheet Pile Wall Used in Model Experiments

2.1. Properties of the Sand. The sand is a dark color river soil and has an angular shape, as shown in Figure 1. Internal friction angles (ϕ) of this material are determined by using a direct shear test, ranging from 42° to 52°, which means the sand has high friction. The material properties of the sand were determined according to ASTM standards [37]; ASTM (D422 - 63 [38]e2, 2007). The granulometry of the soil used in the tests was shown in Figure 2.

Since SPW is driven into the soil with vibration, it cannot be applied on extremely stiff soils. Therefore, in order to provide similarity between field conditions and laboratory model, 16% relative density sand, which is quite loose, was used in the tests. Another objective here was to clearly see the effect of changes in dimensions without changing the properties of the soil. The relative density of the sand was determined according to the ASTM standard [39]. The shear box test on sand was performed according to the ASTM standard [40] and the internal friction angle value was obtained. The material properties obtained are given in Table 1.

The angle of repose of normal loose sand and sand–gravel mixtures is 25–32° [41]. Experiments were carried out to determine the angle of repose of the sand. As a result of this experiment, it was determined that the angle of repose was 35°, as shown in Figure 3. Since the structure of the grains of this sand is angular and basaltic origin, it has a higher internal friction angle than other sands.

2.2. Properties of the SPW. Plexiglass was used as SPW in the tests. The reasons for the use of this material are that its strength and physical properties are standard, and its technical properties are well known. Another reason for using plexiglass as SPW material is that it is easily cut into the desired dimensions, light, and has the capacity to provide the desired deformation in the test. It is known that vinyl material with properties similar to the plexiglass is used in field applications. The material properties of the plexiglass used in the tests can be seen in Table 2.

3. Laboratory Test Procedure and Results

3.1. Test Setup. Due to the stress-dependent soil properties, it is important to accurately model the prototype stress conditions in small-scale modeling experiments. One of the common ways to apply gravity (g) in model experiments is to re-establish the full-size stress levels. Details of the rules and modeling practice used in laboratory modeling can be found in [42]. Information about the scaling laws used in this study is given in Table 3.

Figure 4 and 5 show the plan and cross-sectional view of the modeling test system. In this study, the in-plane strain condition of the test system is assumed. The conditions and dimensions of the test system are similar in all experiments with the exception of the SPW restraint conditions. Since the experimental system is symmetrical, only half of the system is modeled. While the deep of excavation was 100, 150, 200 mm (The equivalent in the prototype was 10, 15, and 20 m), respectively, the thickness of the sand layer was constant and 950 mm (The equivalent in the prototype was 95 m). The width of the system was 750 mm (The equivalent in the prototype was 75 m). According to the scaling law
given in Table 3, experiment-making procedures were explained and results obtained from the experiments were given. All abbreviations used in the study are given in Table 4.

3.2. Model SPW and Properties. The model SPW was made of 3 mm thick plexiglass sheet and consisted of a single piece. In terms of bending stiffness, the model wall is considered nearly equivalent to a prototype-scale reinforced concrete sheet pile wall. The Young’s modulus of these materials, respectively, for plexiglass and concrete is 3.3 GPa and 25 GPa. The ratio of SPW penetration depth to excavation depth is commonly 0.5–2 in engineering application [12, 43, 44]; (100, 150, 200 and 10, 15, 20 m are the model and the prototype dimensions, respectively).

3.3. Procedure of Model Tests. To investigate the behavior of SPWs on sand, model tests were conducted in the laboratory. The first group of test was performed by keeping $H_p$ constant and changing $L$. The second test group was carried out by...
keeping $L$ constant and changing the $H_p$. As a result of these two groups of tests, $\delta_{\text{max}}$ and $q_{\text{ult}}$ were determined.

The appearance of the model test system are as shown in Figures 4 and 5. According to these figures, $H_s$, $H_p$, and $H_z$ are the test tank dimensions, $H_s$ is SPW total length, $e$ is excavation depth, $H_p$ is SPW penetration depth, $L$ is distance of the model foundation to SPW, $B$ is foundation width, $t_w$ is SPW thickness, and $P$ is the load affecting the model foundation. The $P$ point in the foundation is where the loads and deformations were measured. Point $A$ is where $\delta_{\text{max}}$ was measured.

3.4. Scale Effects and Limitations. The loading of the model foundation was carried out with the help of a hydraulic system. Deformation measurements were made at both ends of the foundation and the mid-upper point (point A) of the SPW, as shown in Figures 4 and 5. A $75 \times 75 \times 100$ cm ($H_s$, $H_p$, $H_z$) chamber was used to model the soil environment. A foundation model of $15 \times 15 \times 1$ cm (B) and a plexiglass sheet of $40 \times 75 \times 0.3$ cm ($H_s$, $H_p$, $t_w$) were used during the tests. In model tests, the distance between the test tank wall and the model foundation was designed to be 2B. Similar to the studies performed by Abdelhalim et al. [45]; El Sawwaf and Nazir [46]; Sadrekarimi and Abbasnejad [47], the tank and foundation dimensions were determined by considering that the boundary effects of the tank should be minimal. Thus, in semi-infinite environment conditions, the fundamental rule of model experiments were provided.

The test tank was filled with raining method from a height of 20 cm before placing SPW on the test set. After the filling process was completed, the front part of SPW was excavated until the desired penetration depth was achieved, and loading was started after the model base was placed, as shown in Figure 6 and 7. The model foundation was loaded at a speed of about 2.00 mm/min in the tests. The condition of the model foundation and SPW after loading is shown in Figure 8 in order to determine $\delta_{\text{max}}$, $\delta_{\text{max}}$ was taken from point $A$ Figures 4 and 5).

3.5. First Group Test Results. In the experiments, model foundations were loaded with a speed continuous of 1.0 mm/ min until a settlement value of 0.1 B consisted. The bearing capacity of the foundation is determined when the deformation reaches 10% of the foundation width, which is 15 mm. The load corresponding to this foundation settlement was determined from the graph, and the bearing capacity of the foundation was determined in this way. This method of determining the $q_{\text{ult}}$ has usually been used in model experiments [48, 49]. As a result of the loading tests, the load-settlement graphs for the foundation are obtained in Figure 9.

For $H_p = 20, 25, 30$ cm, the $\delta_{\text{max}}$ versus $L$ graph is shown in Figure 10. For cases of $H_p = 20, 25, 30$ cm, SPW had minimum deformations when $L$ was equal to 4 B/3. This $\delta_{\text{max}}$ was chosen as a reference, and all results were compared with $L = 4B/3$ to see the effect of $L$.

In the case of $H_p = 20$ cm, the decrease of $L$ from 4 B/3 to B, it is determined at $\delta_{\text{max}}$ an increase of 85.74%. The decrease of $L$ from 4 B/3 to 2 B/3 is caused at $\delta_{\text{max}}$ an increase of 141.30%. In case of $H_p = 25$ cm, the decrease of $L$ from 4 B/3 to B is caused at $\delta_{\text{max}}$ an increase of 84.69%. The decreasing of $L$ from 4 B/3 to 2 B/3 it is caused at $\delta_{\text{max}}$ by an increase of 124.72%. In case of $H_p = 30$ cm, the decrease of $L$ from 4 B/3 to B is caused at $\delta_{\text{max}}$ by an increase of 61.13%. The decrease of $L$ from 4 B/3 to 2 B/3 is caused at $\delta_{\text{max}}$ by an increase of 97.23%. As seen in the results, $\delta_{\text{max}}$ decreases with increasing $L$. When the $H_p$ increases; however, the effect of the change of $L$ on $\delta_{\text{max}}$ decreases.

As a result of the tests performed, the effect of the foundation load and the change of $L$ value on $\delta_{\text{max}}$ was measured as in Figure 11, for $H_p = 20$.

As a result of the tests performed, the effect of changing in $L$ on $q_{\text{ult}}$ was measured as shown in Figure 12, for $H_p = 20, 25, 30$ cm. At these 3 penetration depths, in the case of $L = 4B/3$, the SPW had the minimum deformation and the foundation carried the maximum load. However, when $L$ decreased, $\delta_{\text{max}}$ increased, and the load carried by the foundation decreased.
Due to $q_{ult}$ is minimum at $L = 2B/3$, this $q_{ult}$ value was chosen as a reference, and the other $q_{ult}$ values have been compared with this value to see the effect of $L$. In the case of $Hp = 20$, 25, 30 cm.

Figure 9: Load vs. foundation settlement in case of $Hp = 25$ cm.

Figure 10: Changes of $\delta_{max}$ vs. $L$ in the cases of $Hp = 20$, 25, 30 cm.

Figure 11: $q_{ult}$ vs. $\delta_{max}$ relation in case of $Hp = 20$ cm.

Figure 12: Changes of $q_{ult}$ vs. $L$ in case of $Hp = 20$, 25, 30 cm.
$H_p = 20$ cm, the decrease of $L$ from $4B/3$ to $B$ is caused at $q_{ult}$ a decrease of 16.55%. The decrease of $L$ from $4B/3$ to $2B/3$ is caused at $q_{ult}$ a decrease of 9.26%. In case of $H_p = 25$ cm, the decrease of $L$ from $4B/3$ to $B$ is caused at $q_{ult}$ a decrease of 14.04%. The decrease of $L$ from $4B/3$ to $2B/3$ is caused at $q_{ult}$ a decrease of 14.10%. In case of $H_p = 30$ cm, the decrease of $L$ from $4B/3$ to $B$ is caused at $q_{ult}$ a decrease of 4.69%. The decrease of $L$ from $4B/3$ to $2B/3$ is caused at $q_{ult}$ a decrease of 4.98%. As seen in the results, $q_{ult}$ increases with the increase in $L$. When $H_p$ increases, the effect of the change in $L$ on $q_{ult}$ decreases.

3.6. Second Group Test Results. For $L = 2B/3$, $4B/3$, $δ_{max}$ versus $H_p$ are shown in Figure 13. For cases of $L = 2B/3$, $4B/3$, SPW had minimum deformations when $H_p$ was $30$ cm. This $δ_{max}$ was chosen as a reference and all results were compared with the case of $H_p = 30$ cm to see the effect of $H_p$. In case of $L = 4B/3$, the decrease of $H_p$ from $30$ cm to $25$ cm is caused at $δ_{max}$ an increase of 50.90%. The decrease of $H_p$ from $30$ cm to $20$ cm is caused at $δ_{max}$ an increase of 61.13%. In case of $L = B$, the decrease of $H_p$ from $30$ cm to $25$ cm is caused at $δ_{max}$ an increase of 72.97%. The decrease of $H_p$ from $30$ cm to $20$ cm is caused at $δ_{max}$ an increase of 85.74%. In case of $L = 2B/3$, the decrease of $H_p$ from $30$ cm to $25$ cm is caused at $δ_{max}$ an increase of 71.93%. The decrease of $H_p$ from $30$ cm to $20$ cm is caused at $δ_{max}$ an increase of 97.13%. As seen in the results, $δ_{max}$ decreases with increasing $H_p$. When $L$ increases, however, the effect of the change of $H_p$ on $δ_{max}$ decreases.

The effect of load and the change of $H_p$ value on $δ_{max}$ can be seen in Figure 14 for the $L = 2B/3$.

As a result of the tests performed, the effect of the change of $H_p$ value on $q_{ult}$ was measured as in Figure 15, for $H_p$ values where $L = 2B/3$, $4B/3$. At these $L$ values, in the case of $H_p = 30$ cm, the SPW had been the minimum deformation and the foundation carried the maximum load. When $H_p$ decreases, $δ_{max}$ increases, and $q_{ult}$ decreases.

Due to $q_{ult}$ is minimum at $H_p = 20$ cm, this $q_{ult}$ was chosen as reference and the other $q_{ult}$ values have been compared with this value to see the effect of $H_p$. In case of $L = 2B/3$, the increase of $H_p$ from $20$ cm to $25$ cm is caused at in $q_{ult}$ an increase of 20.42%. The increase of $H_p$ from $20$ cm to $30$ cm is caused at in $q_{ult}$ an increase of 51.76%. In case of $L = B$, the increase of $H_p$ from $20$ cm to $25$ cm is caused at in $q_{ult}$ an increase of 17.82%. The increase of $H_p$ from $20$ cm to $30$ cm is caused at in $q_{ult}$ an increase of 36.31%. In case of $L = 4B/3$, the increase of $H_p$ from $20$ cm to $25$ cm is caused at in $q_{ult}$ an increase of 6.78%. The increase of $H_p$ from $20$ cm to $30$ cm is caused at in $q_{ult}$ an increase of 24.47%. As seen in the results, $q_{ult}$ increases with the increase in $H_p$. However, when $L$ increases, the effect of the change in $H_p$ on $q_{ult}$ decreases.

3.7. Outcome of the Model Tests. The load-deformation data of SPW and the settlement-load data of the foundation were given according to different $H_p$ and $L$. The effects of different $H_p$ and $L$ on $δ_{max}$ and $q_{ult}$ were analyzed by comparing the data obtained from the first and second group tests. The $δ_{max}$ corresponding to the maximum $q_{ult}$ obtained in the tests is given in Table 5. The results of $q_{ult}$ are given in Table 6.
In numerical study, the effect of $H_p$ and $L$ on the behavior of SPW and $q_{ult}$ was investigated by using FEM. The data obtained were compared to those obtained from the model test. The Plaxis 3D Foundation program was used for numerical simulations performed in this study \[50\]. In Figure 16, it can be seen that a typical 3D mesh for which a numerical model was created. For 3D analyses, a horizontal midsize mesh and a vertical midsize mesh were used to achieve a balance between the processing time and accuracy.

Similar to the previous studies, the Mohr-Coulomb material model was used in the modeling of the sand \[51–59\]. In all cases, the cohesion was assumed to be 0.01 kN/m$^2$. The modulus of elasticity of the soil was taken as $E = 21000$ kN/m$^2$.

### 4. Numerical Analysis

In numerical study, the effect of $H_p$ and $L$ on the behavior of SPW and $q_{ult}$ was investigated by using FEM. The data obtained were compared to those obtained from the model test. The Plaxis 3D Foundation program was used for numerical simulations performed in this study \[50\]. In Figure 16, it can be seen that a typical 3D mesh for which a numerical model was created. For 3D analyses, a horizontal midsize mesh and a vertical midsize mesh were used to achieve a balance between the processing time and accuracy.

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In the 3D FEM, the sand was modeled with 10-node tetrahedral elements. 10-node tetrahedral elements provide a second-order interpolation of displacement. The SPW was modeled with 6-node triangular surface element with three translation degrees of freedom per node ($U_x$, $U_y$, and $U_z$). The shear modulus ($G$) and bulk modulus ($K$) were obtained using the equations (1) and (2), respectively. The $\nu$ in the formulas is the Poisson’s ratio.

\begin{equation}
G = \frac{1}{2} \left( \frac{E}{(1 + \nu)} \right),
\end{equation}

\begin{equation}
K = \frac{1}{3} \left( \frac{E}{(1 - 2\nu)} \right).
\end{equation}

The properties of sand, SPW, and foundation used in the numerical analysis are shown in Table 7–9, respectively.

In Section1 of the numerical modeling, sand was placed in the soil test tank. Then SPW was driven into sand. The

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**Table 5: Maximum $\delta_{max}$ obtained from model experiments.**

| Penetration depth | Sheet pile - foundation distances (L) $L = 2B/3$ (mm) | $L = B$ (mm) | $L = 4B/3$ (mm) |
|-------------------|-----------------------------------------------|--------------|-----------------|
| $H_p = 20$ cm     | 7.71                                          | 5.93         | 3.19            |
| $H_p = 25$ cm     | 6.72                                          | 5.52         | 2.99            |
| $H_p = 30$ cm     | 3.91                                          | 3.19         | 1.98            |

**Table 6: Maximum $q_{ult}$ obtained from model experiments.**

| Penetration depth | Sheet pile - foundation distances (L) $L = 2B/3$ (kN) | $L = B$ (kN) | $L = 4B/3$ (kN) |
|-------------------|-----------------------------------------------|--------------|-----------------|
| $H_p = 20$ cm     | 0.472                                         | 0.550        | 0.624           |
| $H_p = 25$ cm     | 0.567                                         | 0.648        | 0.667           |
| $H_p = 30$ cm     | 0.716                                         | 0.749        | 0.777           |

**Table 7: Plaxis 3D input parameters of sand.**

| Parameter                  | Input value for Dr = 16% sand |
|----------------------------|-------------------------------|
| Cohesion (kN/m$^2$)        | 0.001                         |
| Unsaturated unit weight (kN/m$^2$) | 14.89                      |
| Modulus of elasticity (kN/m$^2$) | 21000                      |
| Friction angle ($^\circ$)  | 40                            |
| Poisson’s ratio            | 0.307                         |

**Table 8: Plaxis 3D input parameters of Sheet Pile.**

| Parameter                  | Input value |
|----------------------------|-------------|
| E ($E_1$, $E_2$, $E_3$) (kN/m$^2$) | 3299000     |
| Poisson’s ratio ($\nu_{12}$, $\nu_{13}$, $\nu_{23}$) | 0.3         |
| Shear modulus ($G_{12}$, $G_{13}$, $G_{23}$) (kN/m$^2$) | 1222000     |

**Table 9: Plaxis 3D input parameters of foundation.**

| Parameter                  | Input value |
|----------------------------|-------------|
| Dimensions (cm)            | 15 × 15 × 1 |
| Young’s modulus ($E_1$, $E_2$, $E_3$) (kN/m$^2$) | 210         |
| Poisson’s ratio ($\nu_{12}$, $\nu_{13}$, $\nu_{23}$) | 0.3         |
| Shear modulus ($G_{12}$, $G_{13}$, $G_{23}$) (kN/m$^2$) | 79          |
excavation was carried out according to $H_p$ in Section 2. In Section 3, the foundation was placed, the load was applied, and the analyses were performed. Finite element models of the wall and foundation were created by using the foundation modeling module and wall modeling modules in Plaxis 3D. After that finite element analyses were made. In the model tests, $\delta_{max}$ was taken from point A (mentioned in section 3.1) of SPW. It is clearly seen in Figure 17 that numerical solutions also support this situation.

Table 10: Maximum $\delta_{max}$ obtained from numerical analysis.

| Penetration depth ($H_p$) | Foundation (cm) distance (L) |
|--------------------------|-------------------------------|
|                          | $L = 2B/3$ (mm) | $L = B$ (mm) | $L = 4B/3$ (mm) |
| 20                       | 7.58            | 5.88          | 2.33           |
| 25                       | 6.17            | 5.65          | 2.76           |
| 30                       | 3.90            | 2.99          | 1.96           |

Figure 18: $\delta_{max}$ in case of $H_p = 20$ cm, $L = B$ cm and $H_p = 20$ cm, $L = 4B/3$ cm, respectively.

Figure 19: Comparison of data from numerical analyses and laboratory experiments.

Figure 20: Comparison of data from numerical analyses and laboratory experiments.
obtained from the model tests were applied on the foundation in the numerical analyses and $\delta_{\text{max}}$ was determined. The $\delta_{\text{max}}$ obtained as a result of the numerical analyses can be seen in Figures 18(a) and 18(b) and Table 10.

The results obtained from the numerical analyses for $\delta_{\text{max}}$ values were compared with the results obtained from the model tests, and the comparison can be seen in Figures 19 and 20.

5. Conclusions

In this research, SPW deformations ($\delta_{\text{max}}$) and the bearing capacity of the foundations ($q_{ult}$) close to SPW were investigated by laboratory experiments and numerical analysis.

(i) When penetration depth ($H_p$) increases, $\delta_{\text{max}}$ decreases on average by 81% and $q_{ult}$ increases on average by 27%.

(ii) When penetration depth ($H_p$) decreases, $\delta_{\text{max}}$ increases on average by 44%, and $q_{ult}$ decreases on average by 38%.

(iii) As the distance between the foundation and the retaining structure ($L$) increases, $\delta_{\text{max}}$ decreases by approximately 121% and $q_{ult}$ increases by approximately 16%.

(iv) As the distance ($L$) between the foundation and the retaining structure decreases, $\delta_{\text{max}}$ increases by an average of 55%, and $q_{ult}$ decreases by an average of 19%.

(v) Numerical and experimental studies prove that $L$ has great effect on $q_{ult}$ and $\delta_{\text{max}}$ than $H_p$. However, both $L$ and $H_p$ do not have much effect on $\delta_{\text{max}}$ and $q_{ult}$ after a certain distance.

(vi) SPWs should be designed by determining the optimum point for $L$ and $H_p$ values if the field conditions are suitable.

(vii) The results of the Plaxis analyses are more than 90% compatible with the experimental results.

In the present study, the maximum value of $L$ was taken as 4 B/3 due to the geometric conditions of the test system. It is recommended that the effect of $L$ on $\delta_{\text{max}}$ and $q_{ult}$ can be determined in more detail when different $L$ values are tried in future studies.

Data Availability

The data used to support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that they have no conflicts of interest regarding the publication of this paper.

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