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Numerical investigation of the failure of a building in Shanghai, China

Jinchun Chai, Shuilong Shen, Wenqi Ding, Hehua Zhu, John Carter

Abstract

The overturning failure of a 13 storey residential building in Shanghai, China, has been investigated by plane strain finite element analysis (FEA). The results of the FEA indicate that ultimate failure of the building was probably initiated by the formation of tensile cracking in the reinforced concrete piles located under the side of the building adjacent to an excavation. This eventually led to complete structural failure of the piles located along the excavation side, which probably caused further settlement of the building, leading eventually to a toppling failure resulting in overturning of the entire building. Excessive tensile stress in the piles was probably caused by the combination of excavation of soil at one side of the building and the temporary dumping of the excavated soil on the opposite side of the building. It is likely that the effect of temporary dumping of the excavated soil adjacent to the building was either not considered or not properly taken into account in the foundation design nor the construction operations. A simple but important lesson to be draw from this failure is the need for engineers who design foundations in soft soil regions to consider not only the final loading conditions, but also any temporary and transient loading conditions during the construction process.

1. Introduction

A residential complex has been developed at Minhang District, Shanghai, China, which consists of 11 residential buildings, each 11–13 stories, with a total constructed floor area of about 85,000 m². A plan layout of part of the project, including the building of interest in this study (Building No. 7) is shown in Fig. 1. After construction of the superstructure of Building No. 7 was complete, and during excavation and construction of an underground parking space at the southern side of Building No. 7, the building overturned during a short period of time in the direction of the excavation (Fig. 2) at around 17:30 pm Beijing time on June 27, 2009 [18]. After the failure, it was apparent that the building superstructure had rotated predominantly as a rigid body and was largely intact (although resting on its side), implying the possibility of foundation failure. Pictures of the failed building were posted on the internet, causing quite a shock in Chinese society and elsewhere. Access to the building site was subsequently restricted by the local authorities and no forensic investigation report on the failure has been released. Observations made from the picture taken from an adjacent area (Fig. 2) indicate that most if not all of the foundation piles had ruptured at about 1–3 m from the base of the foundation beam to which, presumably, they were originally attached. However, to date detail on: (1) where the failure initiated, for example by failure of the retaining structure for the excavation or the foundation piles; and (2) what was the trigger for or cause of the failure, perhaps the excavation or the dumping of the excavated soil at the northern side of the building; are not clear. In particular, what lessons geotechnical engineers can learn from this failure warrants further investigation.

In this study, the building construction and parking space excavation processes have been simulated using plane strain finite element analysis (FEA), based on known information and after making some assumptions, each of which is clearly identified in the text. By examining the deformations of the foundation and the stresses predicted to act in the retaining structure and the foundation piles, a possible mechanism of failure has been revealed and the likely cause of the overall failure identified. Some of the lessons learned from this study are also discussed.

2. Soil profile

Yin and Xu [15] reported a typical profile of the tip resistance in a cone penetration test (CPT) carried out at the site of Building No. 7, as shown in Fig. 3. Down to the sand used as a bearing layer for...
the piles, the soil profile can be divided into 7 sub-layers. However, no values of the fundamental mechanical properties of these deposits have been reported in the literature. Ma et al. [6] reported a similar soil profile at another excavation site in Shanghai, and some of the mechanical and index properties at that site are given in Fig. 4. By comparing CPT results in Figs. 3 and 4, it can be concluded that the soil profiles at the two sites are similar. Therefore, the data in Fig. 4 will be referred to in determining soil parameters at the site of the failed Building No. 7.

3. Foundation, superstructure and excavation retaining system

3.1. Foundation

Unless otherwise specified, all information reported here about the foundation and the building superstructure is taken from Yin and Xu [15]. The foundation consisted of a pile-beam structure. 114 piles were used in total and the length of each pile was nominally 33 m, i.e., each pile penetrated into the fine sand layer, as shown in Fig. 3. The tops of the piles were at an elevation 2.0 m below the ground surface (personal communication with Dr. J. Yin 2011). The piles were made of pre-stressed concrete and steel bar reinforcement with an annular cross-section. The outside diameter of the pile was 0.4 m and the inner diameter was 0.24 m, so that the thickness of the annular concrete wall of the pile was 80 mm. These piles are referred to as ‘PHC AB’ in the design code of Shanghai, and the arrangement of the steel reinforcement is typically as shown in Fig. 5 [11]. The design properties of the pile are summarised in Table 1.

There is no published detailed information about the structure of the foundation beam cast on top of the piles. In the following analysis it is assumed that the beam had a cross-sectional area of 1.5 m (width) by 2.0 m (height). Based on the information from Yin and Xu [15] and Zhu et al. [18], the foundation of Building No. 7 had plan dimensions of about 45 m in length by 12 m in width.

3.2. Superstructure

Building No. 7 had 13 stories and a cast-in-place reinforced concrete structure. In Shanghai, the height of one storey of a residential building is typically about 2.98 m (2.8 m headroom and about 0.18 m thick floor slab), so that the total height of the building would have been about 38.74 m. In China, for residential buildings with a cast-in-place reinforced concrete structure, the pressure exerted on the foundation by the superstructure is typically in the range from 10 to 15 kPa per storey [4]. Since no detailed information about the superstructure was available, a pressure of 12 kPa per storey has been assumed in this study, implying that the total pressure applied to the foundation by the entire building would be about 156 kPa.

3.3. Excavation retaining structure

The excavation at the southern side of Building No. 7 (see Fig. 1) was supported by a retaining wall formed by cement deep mixing. The width of the wall was about 0.7 m and the depth was about 10 m (personal communication with Dr. J. Yin 2011). The wall was further supported by 6–9 m long soil–nails. However, there is no detailed information available about the cross-section, nor the layout of the soil–nails. Ma et al. [6] reported a case history of an excavation in Shanghai with a similar support system. In Shanghai, generally the soil–nails are formed by a steel tube with a rough outer surface and an outer diameter of about 48 mm and inner diameter of about 41 mm. The spacing between the soil–nails is typically about 1.0 m. Each soil–nail is grouted in place using cement slurry with a water/cement ratio of about 0.5 under an injection pressure of 0.4–0.6 MPa. The soil–nails are generally inclined downward at an angle of 6–10° to the horizontal. By referring to the case reported by Ma et al. [6], it is assumed for the case considered here that there were 4 rows of soil–nails with a
horizontal and vertical spacing of 1.0 m, and an angle between the axis of the soil–nails and the horizontal of 10°. It is further assumed that the length of the nails in the first and third rows was about 9.0 m, while it was assumed that those in the middle row were only 5.5 m long. For the problem geometry studied here, the maximum length of nail that can be simulated without connecting the soil nails to the foundation piles was approximately 5.5 m.

4. Finite element modelling

4.1. Mesh

Based on the information described above, the cross-section A-A (indicated in Fig. 1) is shown in Fig. 6, which forms the plane of analysis in the plane strain model adopted in this study. In the finite element model the foundation soil was represented by 8 noded quadrilateral consolidation elements with pore water pressure degrees of freedom at the 4 vertex nodes only. The superstructure and the temporary embankment (consisting of excavated soil) at the northern side of Building No. 7 (Fig. 1) were represented by 8 noded quadrilateral elements without pore water pressure degrees of freedom. The mesh adopted is shown in Fig. 7 with the displacement and drainage boundary conditions indicated. At the left and right-hand boundaries, the horizontal displacement was fixed, and at the bottom boundary, both horizontal and vertical displacements were fixed. The left and the right-hand boundaries were considered to be impermeable, and the ground surface and the bottom boundaries were considered permeable, i.e., they were drainage boundaries. However, the area of ground surface under the temporary embankment was assumed to be impermeable. The surface formed by the excavation was treated as a drainage boundary.

4.2. Modelling methods, constitutive models and parameters

4.2.1. Foundation soils

The clayey soils were modelled as Modified Cam Clay (MCC) [10], and the sand layers were represented by linear elasticity. In the case of the MCC model, in the elastic range of behaviour Young’s modulus was adopted as the usual function of mean effective stress ($p_0$) and voids ratio ($e$), except when the effective mean stress was less than 100 kPa. In the latter case a value of Young’s modulus corresponding to $p_0 = 100$ kPa was assumed. This option was adopted to avoid the possibility of predicting excessive heaving of the clayey soil layers due to the excavation-induced unloading.

The values of the model parameters adopted in the analysis are listed in Table 2. Except for the values of hydraulic conductivity ($k$), all values were assessed by referring to the data in Fig. 4 [6], as well as relying on experience. The initial values of $k$ were determined by referring to the ranges recommended in the Shanghai Site Investigation Code [12]. During analysis of the consolidation process, the values of $k$ were allowed to vary with voids ratio ($e$) according Taylor’s [14] equation.

$$k = k_0 \cdot 10^{-(e_0 - e)/C_e}$$  (1)

where $k_0$ = initial hydraulic conductivity, $e_0$ = initial void ratio, $k$ = current hydraulic conductivity, $e$ = current void ratio, and $C_e$ = a constant assigned a value of 0.4$e_0$ in this study. In Shanghai, the groundwater level is about 0.5–1.0 m below the ground surface,
and in the present analysis it was assumed to be 0.5 m below the ground surface. The adopted profiles of initial effective stresses, overconsolidation ratio (OCR), yield loci \((p_y)\) and undrained shear strength \((S_u)\), as predicted by the MCC model, are listed in Table 3.

### 4.2.2. Piles

It was assumed that the 114 piles were arranged in three (3) rows, with each row having 38 piles in it. The building was assumed to be 45 m long, so that the spacing between adjacent piles was about 1.2 m. Based on the cross-sectional geometry shown in Fig. 5, and adopting a Young’s modulus \((E)\) for the concrete of \(2.05 \times 10^7\) kPa and \(2.05 \times 10^8\) kPa for the steel reinforcement, the area weighted average value of \(E\) for the pile section is about \(2.18 \times 10^7\) kPa. Thus the bending rigidity \((EI)\) of a single pile with respect to the diameter, e.g., the \(x\) or \(y\) axis in Fig. 5, can be evaluated as about \(2.4 \times 10^4\) kN m². Poisson’s ratio of the piles

![Fig. 6. Cross-section along section A-A.](image)

![Fig. 7. Finite element mesh adopted.](image)

Table 3

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>(\sigma_{\text{ho}}) (kPa)</th>
<th>(\sigma_{\text{om}}) (kPa)</th>
<th>OCR</th>
<th>(S_u) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>5.0</td>
<td>0.0</td>
<td>13.0</td>
<td>22.4</td>
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<tr>
<td>0.5</td>
<td>5.7</td>
<td>9.6</td>
<td>13.0</td>
<td>22.4</td>
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<td>2.5</td>
<td>16.9</td>
<td>28.2</td>
<td>3.9</td>
<td>22.4</td>
</tr>
<tr>
<td>3.5</td>
<td>18.0</td>
<td>36.0</td>
<td>2.8</td>
<td>21.8</td>
</tr>
<tr>
<td>7.5</td>
<td>32.0</td>
<td>64.0</td>
<td>2.0</td>
<td>21.8</td>
</tr>
<tr>
<td>13.5</td>
<td>52.4</td>
<td>104.8</td>
<td>2.0</td>
<td>35.7</td>
</tr>
<tr>
<td>21.5</td>
<td>85.2</td>
<td>170.4</td>
<td>1.5</td>
<td>62.1</td>
</tr>
</tbody>
</table>

4.2.2. Piles

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Table 2

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (m)</th>
<th>(E) (kPa)</th>
<th>(v)</th>
<th>(k)</th>
<th>(i)</th>
<th>(M)</th>
<th>(e_0)</th>
<th>(\gamma) (kN/m³)</th>
<th>(k_h) ((10^{-8}) m/s)</th>
<th>(k_v) ((10^{-8}) m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silty clay</td>
<td>0–2.5</td>
<td>–</td>
<td>0.15</td>
<td>0.008</td>
<td>0.08</td>
<td>1.2</td>
<td>0.8</td>
<td>19.1</td>
<td>4.04</td>
<td>2.02</td>
</tr>
<tr>
<td>Clay</td>
<td>2.5–3.5</td>
<td>–</td>
<td>0.15</td>
<td>0.008</td>
<td>0.08</td>
<td>1.2</td>
<td>1.14</td>
<td>17.6</td>
<td>0.42</td>
<td>0.21</td>
</tr>
<tr>
<td>VSC-1’</td>
<td>3.5–7.5</td>
<td>–</td>
<td>0.15</td>
<td>0.035</td>
<td>0.33</td>
<td>1.0</td>
<td>1.38</td>
<td>16.8</td>
<td>4.0</td>
<td>2.0</td>
</tr>
<tr>
<td>VSC-2</td>
<td>7.5–13.5</td>
<td>–</td>
<td>0.15</td>
<td>0.034</td>
<td>0.34</td>
<td>1.0</td>
<td>1.45</td>
<td>16.6</td>
<td>3.0</td>
<td>1.5</td>
</tr>
<tr>
<td>Clayey silt</td>
<td>13.5–21.5</td>
<td>–</td>
<td>0.15</td>
<td>0.018</td>
<td>0.18</td>
<td>1.2</td>
<td>1.03</td>
<td>18.0</td>
<td>6.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Clayey silt</td>
<td>21.5–30.0</td>
<td>–</td>
<td>0.15</td>
<td>0.008</td>
<td>0.08</td>
<td>1.2</td>
<td>0.75</td>
<td>19.3</td>
<td>10.0</td>
<td>5.0</td>
</tr>
<tr>
<td>Clayey silt</td>
<td>30.0–33.0</td>
<td>–</td>
<td>0.15</td>
<td>0.006</td>
<td>0.06</td>
<td>1.2</td>
<td>0.72</td>
<td>19.5</td>
<td>10.0</td>
<td>5.0</td>
</tr>
<tr>
<td>Fine sand</td>
<td>33.0–50.0</td>
<td>30,000</td>
<td>0.1</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>0.63</td>
<td>20.0</td>
<td>500.0</td>
<td>500.0</td>
</tr>
<tr>
<td>Fill</td>
<td>–</td>
<td>15,000</td>
<td>0.3</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>17.0</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

Notes: (1) ‘Very soft clay.
(2) \(e_0\) = initial void ratio; \(k\) = hydraulic conductivity; \(\kappa\) = slope of unloading–reloading curve in \(e\text{–}\ln p_0\) plot (\(e\) is void ratio and \(p_0\) is consolidation pressure); \(i\) = slope of virgin loading curve in \(e\text{–}\ln p_0\) plot; \(M\) = slope of critical state line (CSL) in \(p\text{–}q\) plot (\(q\) is deviator stress), and \(e_0\) = void ratio corresponding to \(p_0 = 1\) kPa and on CSL in \(e\text{–}\ln p_0\) plot.
Interaction between the foundation piles and the soft soil is a three-dimensional (3D) problem and when analysing it with a two-dimensional (2D) model some shortcomings will be inevitable, such as: (a) the possibility of relative horizontal movement occurring between the piles and soil cannot be simulated; and (b) equivalent values of the bending rigidity $EI$ (where $E$ is Young's modulus and $I$ is the second moment of area) and equivalent values of $EA$ (where $A$ is the cross-sectional area) of the pile section cannot be satisfied simultaneously. In this study, it is considered that the bending rigidity of the pile is more important and so equivalent values of $EI$ have been modelled in the 2D analysis. However, in interpreting the numerical results, the effect of the difference of the modelled and actual cross-sectional areas of the pile on the simulated stresses in the pile has been considered. Along the longitudinal direction of the building the value of $EI$ for a row of piles would be about $2.0 \times 10^8$ kN m$^2$/m, and the deduced thickness of the pile-wall will be about 0.222 m, as illustrated in Fig. 8. However, if the bending moment ($M_y$) and the second moment of area ($I_y$) with respect to the $x$ axis are the same, then the maximum tensile stress in the pile is linearly related to the maximum distance to the $x$ axis (i.e., the distance in the $y$ direction). Therefore, for given values of $M_y$ and $I_y$, and for the conditions given in Fig. 8, the maximum tensile stress in the pile with an annular cross-section is estimated to be about 1.8 times that predicted in the rectangular-shaped continuous pile-wall.

The 0.222 m thick plane strain pile-wall was simulated using 2 rows of 8 noded elements with pore water pressure degrees of freedom. Tensile yielding of the piles was considered, with the tensile strength of the concrete set as 1/3 of its compressive strength (e.g., [16]) giving a tensile strength of about 6150 kPa. When the induced tensile stress exceeded the strength, the Young's modulus of the pile was reduced to 1/10 of its initial value. As explained above, the tensile stresses induced by a bending moment in an annular pile and a pile-wall are different. Ideally an equivalent tensile strength for the pile-wall should be used in the calculations described here. However, considering other influencing factors such as the pre-stress acting in the pile, the tensile strength of the concrete itself was used here to determine when the Young's modulus should be reduced, and any decision whether the pile failed in tension was based on analysis using the predicted values of stress.

For composite ground, such as a soft soil 'reinforced' by piles, the consolidation process is controlled by both the permeability and the coefficient of consolidation of each individual material component. Therefore, it is not possible to model exactly the consolidation behaviour of soft ground containing discrete piles using a 2D pile-wall model. In the 2D analysis, the pile-wall was modelled by consolidation elements. The pile-wall has a much higher stiffness than the soft soil, and if the same value of permeability as for the soft soil is adopted, the coefficient of consolidation of the wall material will be much higher than actual case. Considering this point, the permeability of the pile-wall was arbitrarily set as 1/10 of the value for the soft soil. The resulting coefficient of consolidation is still higher than that of the soft soil. Clearly this approach is an approximation of the real situation.

### 4.2.3. Superstructure

Since the building rotated onto its side in almost an intact condition, the superstructure was simulated simply as a series of rigid concrete boxes. Young's modulus of the superstructure was assumed as 2.05 × $10^7$ kPa and Poisson's ratio as 0.1. Using a unit weight of the concrete wall of 24 kN/m$^3$, the model as shown in Fig. 6 to result in an applied pressure of about 12.0 kPa/storey, the thickness of the wall as well as the floor slab would be about 0.295 m. Note that by assuming this wall thickness the weight of the transverse walls of the building is also included in the analysis.

### 4.2.4. Retaining system

In Shanghai, normally the amount of cement used in deep mixing construction is about 13% by dry weight, and the confined compression strength ($q_{uc}$) of the soil–cement column formed by mixing is about 500 kPa [2]. It was assumed that the value of $q_{uc}$ of the retaining wall formed by deep cement mixing was 500 kPa and it was further assumed that Young's modulus ($E$) of the column material was 200 times $q_{uc}$ [3], resulting in a value of $E$ of about $10^5$ kPa. This value of $E$ together with a Poisson's ratio $v$ of 0.1 were adopted for the retaining wall. The tensile strength of the wall was assumed to be 50 kPa (1/10 of its $q_{uc}$ value) [13]. The 0.7 m thick soil–cement wall was modelled by 3 rows of 8 noded elements.

The soil–nails for supporting the retaining wall were simulated by using plane strain bar elements without interface elements between the soil–nails and the surrounding soil. It was also assumed that the nails are connected to the foundation piles and the transverse walls of the building. It was also assumed that each soil–nail (steel tube) had a cross-sectional area of $A = 489$ mm$^2$ (outer diameter of 48 mm and inner diameter of 41 mm), with a Young's modulus $E = 2.05 \times 10^8$ kPa, implying a value of $EA$ of about 100 MN. In the field, the soil–nails might have penetrated under the building, but in a plane strain analysis it is not usually possible to place a soil–nail under the building without connecting it to the foundation piles. Analyses for two extreme conditions of connecting and not connecting the first and third rows of soil–nails to the foundation piles were conducted. Adopting the case in which the nails are connected to the foundation piles should provide an "upper bound" on the restraining effect that the nails have on the retaining wall, but it is acknowledged that this case may depart somewhat from the actual situation.

### 4.3. Simulation of construction

Simulation of the building construction (and collapse) was carried out in the following steps:

- **Step 1**: Set-up the initial stress conditions and install the foundation (piles and beams). The foundation was put in place by changing the corresponding material properties of the relevant finite elements without explicitly simulating the pile installation process.
- **Step 2**: Construct the superstructure. There is no detailed field construction record. In Shanghai, for a structure constructed using cast-in-place reinforced concrete, generally the construction speed is about 1 storey per week or less [17]. In the finite element analysis, construction of the superstructure at rate of 1 storey per 10 days was simulated by turning on the gravity force of the corresponding structural elements over the appropriate increment of time.
- **Step 3**: Construct the retaining wall and the soil–nails. For the retaining wall, as for the foundation, only the properties of the corresponding elements were changed and this change...
was assumed over 2 weeks of elapsed time. The length of this
time period has negligible effect on the foundation deformation
because most of the weight of the superstructure was sup-
ported by the piles and there was only very small excess pore
water pressure generated in the soft soil layers. The soil–nails
were introduced at appropriate stages during the excavation
process, e.g., when the excavation reached 1.0 m depth, the first
row of soil–nails was introduced.

- **Step 4:** Conduct the excavation and temporary embankment fill-
ing. As shown in Fig. 1, there was a large excavated area on the
southern side of Building No. 7. The average excavation depth
was about 4.6 m at the time the building collapsed [15]. Since
the excavation was conducted in an open site and the excavated
soil was dumped nearby (on the northern side of Buildings No. 6
and No. 7 (see Fig. 1)), based on local experience it is estimated
that the excavation speed was about 1 m per 2 days (i.e., 0.5 m/
day). In the analysis, it is assumed that the first stage of fill
placement was during excavation from 0 to 2.0 m depth, and the
second stage filling was during excavation from 2.0 to
4.6 m (Fig. 6).
- **Step 5:** Simulate consolidation of the foundation soil. After the
evacuation and temporary placement of embankment filling,
and in the absence of further construction activity, the ongoing
consolidation response of the foundation soils was simulated.

### 4.4. Geometric non-linearity

In this problem it is likely that deformation of the foundation soil
may have caused changes to the initial geometry of the foun-
dation and the superstructure. Especially for the foundation piles,
if the axial load becomes eccentric it will increase the bending mo-
ment acting in the pile. For this reason geometric non-linearity was
considered approximately in the analysis by updating the nodal
coordinates at the end of each incremental step.

### 5. Analysis results

The program used to conduct the analyses described in this
paper is a modified version of the original CRISP program [1]. Three
cases were analysed, as listed in Table 4. Case-1 may be regarded as
a base case, while Case-2 was conducted to investigate the effect of
the length of the soil–nails, and Case-3 was the same as Case-1
except it was assumed that the excavated soil was not dumped at
the northern side of the failed building, i.e., there was no
temporary embankment constructed.

Since the detailed construction history has not been reported it
was assumed that the building collapsed 15 days after the excava-
tion on the southern side had reached a depth of 4.6 m, or in other
words the total time from the start of the excavation to the col-
lapse of the building was 24 days. Unless otherwise mentioned
in the analysis by updating the nodal
coordinates at the end of each incremental step.

To identify the possible trigger of the building failure, firstly the
possibility of bearing capacity failure of the foundation soil under
the temporary embankment has been assessed. Considering
undrained conditions and thus zero friction angle of the subsoil,
a bearing capacity of about 115 kPa (\(= 5.14 s_u\), where \(s_u\) is the
undrained shear strength of the soil near the ground surface with
a value of 22.4 kPa (Table 3)) can be evaluated. When calculated
using the fill thickness, the maximum vertical pressure from the fill
to the foundation soil is about 175 kPa (10.3 m fill with a unit
weight of 17 kN/m\(^3\)), which is larger than the estimated bearing
capacity. However if the trapezoidal fill load is converted to an
equivalent uniformly distributed load, with a loading width of
26 m (the width of the fill in the cross-section simulated), the mag-
nitude of the equivalent uniform load will be about 101 kPa. While,
if it is assumed that the slopes of the fill at both sides are the same
as that on the left-hand (steeper) side, the width will be about
20 m and the equivalent load will be about 110 kPa. In both cases
the magnitudes of these equivalent loads are smaller than the esti-
mated bearing capacity. Further, allowing for the effects of some
partial consolidation, a bearing capacity failure under the fill load
is considered unlikely. The numerical results indicate that at the
end of filling there are just a few constrained zones under the fill
material with a factor of safety about 1.0, but general bearing
capacity failure is not predicted.

Thus from the available evidence it was deduced by the authors
that the possible triggers or causes of the building failure might in-
clude: (a) failure of the retaining structure constructed to support
the southern excavation; and/or (b) failure of the foundation piles.
Accordingly, in presenting the results of the finite element analysis
focus has been placed on the deformation of the retaining structure
and the piles as well as the stresses induced in them.

### 5.1. Deformation of the wall and stresses in the retaining structure

The lateral displacement profiles predicted at the retaining wall
location are compared in Fig. 9. Fig. 9(a) compares the results for
Case-1 at the end of the excavation and 15 days after the excava-
tion to 4.6 m depth. It can be seen that due to dissipation of excess
pore water pressures induced by the embankment construction
and the basement excavation, there is an increase of the lateral dis-
placement with elapsed time. In Fig. 9(b) are shown the results for
Cases-1, 2 and 3. Comparing the results of Cases-1 and 2 indicates
that connecting the soil–nails to the foundation piles has only a
very small effect on the predicted lateral displacement of the wall.
However, comparing the results of Cases-1 and 3 shows clearly
that dumping the excavated soil at the northern side of the build-
ing has a very significant effect. The predicted maximum lateral
displacement for Case-1 (about 181 mm) is about 12 times of that
of Case-3 (about 15 mm).

Using the values of stresses predicted at all 9 integration points
of each element making up the wall, the profiles of vertical effec-
tive stress at both wall surfaces (the excavation and retained soil
sides) were linearly extrapolated using the method illustrated in
Fig. 10. The results of this extrapolation are plotted in Fig. 11(a)
for Case-1 and compared for all three cases in Fig. 11(b).
Fig. 11(a) shows that at the excavation side, at about 5.0 m depth,
some tensile stress was predicted, but the values are smaller than
the specified tensile strength of the wall material of 50 kPa. Hence,
tensile failure of the retaining wall was not predicted. As for the
lateral displacement of the wall, dumping the excavated soil on
the northern side of the building significantly influenced the verti-
cal stresses predicted in the wall (Fig. 11(b)).

The predicted distributions of tensile force in the soil nails are
plotted in Fig. 12. For Case-1 the maximum tensile force is about
43.5 kN and this occurred in the third row of soil–nails (at 3 m
depth). The tensile strength of each soil–nail is estimated to be
about 700 kN, and therefore the finite element predictions indicate
that tensile failure is not likely to occur in these particular soil
nails. Although the pullout failure mechanism was not considered
in the analysis, it is noted from the results shown in Fig. 12 that for
the third row of nails the maximum tensile force is mobilized

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Table 4  
Cases analysed.

<table>
<thead>
<tr>
<th>Case</th>
<th>Temporary embankment</th>
<th>Soil–nail connection condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Yes</td>
<td>Not connected to the pile</td>
</tr>
<tr>
<td>2</td>
<td>Yes</td>
<td>Connected to the pile</td>
</tr>
<tr>
<td>3</td>
<td>No</td>
<td>Not connected to the pile</td>
</tr>
</tbody>
</table>
within the first 3 m of the soil nail, for which an average interface shear stress of about 100 kPa can be estimated. In practice the soil nails were grouted using cement slurry and the outside surface of the soil nail was rough, so that the interface strength between the cement annulus formed around the soil–nail and the soil nail is considered to be generally larger than 100 kPa. In other words, pull out failure of the nails would have been unlikely. In terms of the tensile force in the soil–nails, Case-3 resulted in a smaller predicted maximum value in the first row (at 1.0 m depth) but about the same maximum values for the other rows.

5.2. Lateral displacement, bending moment and tensile stress in the piles

The predicted lateral displacement profiles of the row of piles closest to the excavation are compared in Fig. 13, which shows that the maximum lateral displacement of the pile is slightly smaller than that of the retaining wall (Fig. 9), but the profiles of displacement are similar. Again, the results of Cases-1 and 2 are almost the same, but Case-3, without the temporary embankment, generally has much smaller values. The deformed shape of the piles indicates that high bending moment can be developed in them.

Assuming the vertical stress variation between integration points is linear and using the extrapolation method, as illustrated in Fig. 10, the distributions of bending moment \( M \) in the pile rows have been calculated and these are shown in Fig. 14 for Case-1. The values of \( M \) in this figure have been converted to values of bending moment for each individual pile. Fig. 14(a) shows that the maximum value of \( M \) increases with elapsed time after excavation. The same tendency for an increase in bending moment in the piles was reported by Ong et al. [7] in their centrifuge model tests. From Fig. 14(b), it can be seen that the row of piles closest to the excavation exhibits the largest value of maximum bending moment, Leung et al. [5] conducted laboratory centrifuge model tests to examine the effect of excavation on pile groups, and their results showed a similar tendency. For Case-1, the simulated maximum value of \( M \) is about 66.8 kN m at the end of the excavation, and 91.0 kN m 15 days after the excavation.

Using an ultimate tensile strength of the concrete of 6150 kPa (i.e., 1/13 of 80 MPa), the bending capacity of the annular concrete section is about 34 kN m. The strength of the steel reinforcement is 1420 MPa, allowing a bending capacity of the steel reinforcement of approximately 68 kN m to be evaluated. Thus the overall capacity of the pile section is estimated to be about 102 kN m, which is very close to the design bending capacity \( M_{\text{max}} \) of 104 kN m. This
is close to will also induce greater compression of the pile, which acting in an annular pile due the weight of the superstructure is likely to be larger than 3000 kPa. However, larger values of \( \sigma_t^k \) will also induce greater compression of the pile, which in turn will cause the transfer of more load to the surrounding soil. For example, a compressive stress of 3000 kPa would induce about 4.5 mm of one-dimensional compression of a 33 m long free-standing concrete pile. Therefore, it is not appropriate to simply convert the vertical effective stress acting in the annular pile by the corresponding ratio of the cross-sectional areas of the wall and annular piles. In addition, the compression of the steel reinforcement due to the weight of the superstructure is also likely to reduce the initial pre-stressing effect in the concrete. Thus, in order to avoid undue complexity, a value of \( \sigma_t^k = 3000 \) kPa will be used in following analysis, but keeping in mind the likely possibility that the axial stress acting in an annular pile may be larger than this value.

Another point to be borne in mind is that the predicted bending moments should continue to increase after the time of interest considered here, i.e., 15 days after excavation, and under estimation of the value of \( \sigma_t^k \) should not change the general conclusions reached, as detailed below. If this value of \( \sigma_t^k \) is considered as an additional "pre-stress" acting in the pile, which can potentially increase the tensile cracking bending moment, an additional moment capacity of about 17 kN m is estimated for the pile. Thus overall, the effective tension cracking moment will be about 80 kN m. At the end of excavation, the simulated bending moment is less than 80 kN m, but 15 days after the excavation, the predicted value of \( M \) is larger than 80 kN m, which is much larger than the capacity of the reinforcement alone, i.e., about 68 kN m. These predictions indicate that it is likely the piles would have failed gradually, starting from the row of piles located along the side of the excavation.

A comparison of the bending moments predicted in the row of piles located along the side of the excavation is given in Fig. 15. This figure clearly shows that if the excavated soil had not been dumped at the northern side of the building, then the predicted value of \( \sigma_t^k \) would be less than 11 kN m, with the maximum value occurring near the bottom of the pile. Poulos and Chen [9] proposed a design chart for estimating the maximum bending moment in a single pile induced by a braced excavation in a soft clayey deposit. If the difference in the support systems assumed by Poulos and Chen and that adopted in the case studied here is ignored, and the pile row nearest to the excavation side is treated as a single pile, a maximum bending moment of about 28 kN m can be estimated from Poulos and Chen's design chart, which is larger than the value of 11 kN m simulated by the finite element analysis. Ong et al.'s [8] centrifuge model test results indicate that the maximum bending moment of a pile in a canted pile group is much less than that in a single pile at the same distance behind the excavation boundary. For the conditions investigated, their results show that the maximum bending moment of a pile in a 2 x 3 pile group (3 rows and 2 piles/row) is less than 50% of a single pile at the same distance behind the excavation boundary. Combining the result from Poulos and Chen's chart and the results of Ong et al.'s [8] model test indicates that the small bending moments predicted for Case-3 are probably quite reasonable.

The possibility of pile failure due to the development of tensile stresses in the pile section also warrants checking. Adopting an equivalent rectangular cross-section of the pile, as shown in Fig. 9(b), the predicted maximum tensile stress is about 5990 kPa (Fig. 16). The effect of pre-stress was not considered in the simulation but the axial compressive stress in the pile due to the weight near the ground surface. This value of 3000 kPa was simulated by a plane strain analysis assuming a pile-wall. The cross-section of the representative segment of the pile-wall is larger than (about 3 times) that of the equivalent annular pile. Consequently, the value of \( \sigma_t^k \) acting in an annular pile due the weight of the superstructure is likely to be larger than 3000 kPa. However, larger values of \( \sigma_t^k \) will also induce greater compression of the pile, which in turn will cause the transfer of more load to the surrounding soil.
of the superstructure was taken into account. That means for the rectangular shaped pile, the maximum tensile stress induced by the bending moment is about 8990 kPa. This simulation was conducted under the assumption of equal values of bending rigidity in the plane strain pile-wall and the discrete annular piles. Hence the maximum tensile stress in the actual piles of annular cross-section (Fig. 9(a)) due to bending is approximately 1.8 times the corresponding value predicted for the equivalent rectangular pile-wall, i.e., about 16,182 kPa. The overall tensile capacity of the concrete in the pile cross-section is given by the sum of the tensile strength of the concrete of 6150 kPa, the initial pre-stress in the pile of 5300 kPa and the axial compressive stress acting in the pile due to the weight of the superstructure of 3000 kPa. This overall tensile capacity is therefore approximately 14,450 kPa, which is smaller than the maximum tensile stress of 16,182 kPa predicted to act in the pile section. Hence, these calculations indicate that cracking of the pile due to the tension induced by bending is highly likely.

Given the analysis of the simulation results presented above, it is suggested that the failure mechanism of the building probably involved first the progressive failure in bending of the foundation piles located along the excavation side of the building, which then probably caused additional settlement of this side of the building and eventually led to tilting and the development of a toppling mechanism, ultimately causing overturning and collapse of the building as illustrated in Fig. 17. The fact that almost all piles were broken or ruptured about 1–3 m from the base of the foundation beam to which they were originally connected (Fig. 2) provides direct evidence of pile failure. Furthermore, if the excavated soil had not been dumped at the northern side of the building, the predictions indicate it is highly likely that the building would have remained standing.

5.3. Settlement of the building

The building failed by overturning towards the excavation or southern side of the building, and the question arises whether the finite element model indicated this phenomenon or not. Fig. 18 shows a plot of the simulated settlement curves for both sides of the building. It can be seen that due to the weight of the superstructure alone, the building settled about 27.5 mm. Then, during the excavation and the embankment filling process, the southern or excavation side of the building (labelled as “right side” in Fig. 18) heaved slightly at first and then eventually settled. Meanwhile, the northern side of the building (labelled as “left side”) continuously settled and at the end of the excavation the predicted overall settlement was greater on this northern side (about 6 mm more than the southern side). At this stage, the overturning mechanism was not reproduced because tensile failure of the equivalent pile-wall was not simulated with the adopted rectangular cross-section of the pile.
5.4. Pore water pressure and shear stress

For completeness, the excess pore water pressure distribution predicted in the ground 15 days after the excavation is shown in Fig. 19. Under the centre of the temporary embankment fill, the maximum value of excess pore pressure at this time is still greater than 80 kPa, and at the excavation side the suction pore water pressures are generally smaller in magnitude than 40 kPa.

Contours of the relative shear stress level (SSL), i.e., the ratio of the maximum shear stress acting at any point to the undrained shear strength at that location, are depicted in Figs. 20(a) and (b) for Case-1 and Case-3 for the ground on the excavation side of the building. It should be noted that the relative shear stress level is effectively the inverse of the local factor of safety. For Case-1 (Fig. 20(a)), there is a relatively large zone of soil below the excavation surface that has reached a failed state, i.e., SSL = 1.0, but the failure is contained. For Case-3, there are a few spots where SSL = 1.0, but generally the foundation has obviously lower values of SSL (and hence higher values of the factor of safety) than for Case-1, which indicates that the temporary dumping of soil adjacent to the building on its northern side has reduced the local factor of safety of the foundation soil significantly.

6. Discussion

For this case history it is inferred from the finite element analysis that the design of the foundation seemed to be adequate provided there was no temporary dumping of the excavated soil adjacent to the building. It is noted that similar designs have been used successfully in a number of residential buildings in Shanghai. It is possible that the effect of the temporary dumping of the excavated soil adjacent to the building had not been properly considered in the design process. In addition, the contractor and/or field engineer(s) might not have had sufficient knowledge of the behaviour of soft clayey deposits, and perhaps in ignorance dumped the soil on the northern side of the building for convenience, presumably with the aim of saving construction costs.

This case history teaches us a simple but important lesson and that is in order to carry out a foundation design and construction successfully in a soft deposit area, geotechnical engineers not only need to consider the final configuration and loading conditions, but also any possible temporary loadings, such as the temporary stockpiling of construction materials and the storage of excavated soils. In addition, the detailed construction procedures should strictly specify the places for temporary storage of waste soil and other construction materials.
7. Conclusions

The mechanism of overturning of a 13 storey residential building in Shanghai, China, has been investigated by plane strain finite element analysis (FEA). The results of the FEA indicate that the failure probably started by the formation of tension cracks in the concrete of the foundation piles along the southern, excavation side of the building, followed by progressive failure of the pile sections and eventually overturning of the building. The excessive tensile stresses in the piles were caused by the combination of excavation at one side of the building and temporary dumping of the excavated soil at the opposite side of the building. The numerical parametric study indicates that if dumping of the excavated soil adjacent to the building had not occurred, then it is likely that the excavation alone would not have been sufficient to induce failure of the piles and overturning of the building.

A maximum bending moment of approximately 91 kN m was predicted in the piles along the excavation side of the building. This value is larger than the effective design bending capacity in terms of a tensile cracking moment of about 80 kN m (composed of a specified pure bending moment of 63 kN m plus a self-weight induced “pre-stress effect” of about 17 kN m), but less than the specified total bending capacity of approximately 104 kN m. This indicates that the piles probably failed in a progressive manner. It is suggested as likely that the failure started from the formation of tension cracks on the surface of the pile, followed by breakage of the concrete and then yielding and possibly rupture of the steel reinforcement in the piles.

It is evident that for this case history, the effect of the temporary embankment filling adjacent to the building may not have been properly taken into account in the foundation design or in the construction operation. A simple but important lesson is that in order to successfully design a foundation in a soft soil area, engineers not only need to consider the final loading conditions, but also any temporary loadings such as those that might arise from the storage of construction materials and construction generated waste soils.

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References