Research Article

Study on Bearing Capacity of the Existing Engineering Pile Group without Lateral Displacement during Dynamic Top-Down Construction

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The bearing capacity of the vertical underpinning structure system is the key index in the design of top-down construction for adding a basement layer under existing buildings. The influence of the lateral restraint is the most significant under the dynamic construction excavation. For the problem of the bearing capacity of the existing engineering pile group under the top-down construction, the linear eigenvalue stability method was used first to study the influence of the lateral restraints such as the horizontal resistance of soil, the diameter of piles body, and the bending rigidity of the temporary steel bracing on its bearing capacity. The corresponding critical stability load and the effective length coefficient were then obtained. Then, based on the nonlinear extreme point stability method with the initial geometrical imperfection, the amplification range for the effective length coefficient was studied. Finally, based on the current Chinese Code Formula (JGJ 94-2008) and considering the influence of the compression buckling effect of the high cap pile, the present solution of the bearing capacity for the pile body was obtained and compared with the code solution. It turns out that the nonlinear bearing capacity of existing engineering piles group with initial imperfection is smaller than the critical stability load of the linear eigenvalue and increases with the increase of the imperfection amplitude, and the amplification range of the effective length coefficient is $1.10 \sim 1.20$. The present solution of the bearing capacity with the compression buckling effect is $1.10 \sim 1.30$ times of the code solution, which shows that the code solution is partial to safety, and the residual bearing can be properly considered in the design.

1. Introduction

The technology principle and the operation process of the top-down construction are fully used for reference in the construction technology of the underground space in existing buildings, which can be regarded as the extension of the application for the top-down method technology in foundation pit engineering [1–5]. Generally, the overall operation process is as follows: first, the surrounding enclosure structure and the vertical support system are constructed; then, the earthwork excavation is carried out in the process of top-down construction, and the horizontal supporting structures such as underground beams and slabs are constructed at the same time; then, the foundation slab is poured to the base elevation after the excavation; finally, the vertical bearing components such as the basement exterior wall, the column, and the wall are constructed. When the vertical bearing components of the basement reach the design strength, the vertical temporary underpinning structures (such as anchor static pressure pile group, existing
engineering pile group, etc.) within the height range of the new basement should be chiseled out. The operation process is a dynamic construction process, in which the design of the vertical underpinning structure system is one of the key links in the top-down construction technology of existing buildings [6–10].

The application of the top-down construction for adding layers under existing buildings is different from that of the conventional top-down construction. As a result, the load changes of the vertical underpinning structure system in the dynamic construction process are also quite different. The conventional top-down construction is aimed at synchronous construction of the underground and the above ground structures, and the load of the vertical underpinning structure system is gradually increased with the construction process. While the top-down construction for adding a layer under existing buildings is to build a new underground space when the above ground structure has been completed, the upper load must be transferred to the vertical underpinning structure system before the bottom top-down excavation [6, 11–14], which is relatively more difficult to design.

The vertical underpinning structure system is generally composed of the transfer bearing platform and the erect column pile. The erect column pile can adopt an additional anchor static pressure pile or existing engineering pile, and the layout forms include “one-column-one-pile” and “one-column-multi-pile”. The anchor static pressure pile is generally used in the low-rise existing buildings, and it is widely applied under the condition of limited construction space [15–17]. However, for the high-rise existing buildings with the pile foundation, anchor piles are generally added before the excavation. Moreover, together with existing engineering piles, they are used as the vertical support structure system in the construction stage to compensate for the post-load increase and achieve higher bearing capacity. The vertical temporary underpinning technology has high requirements, great difficult and complex joint connection, and there are few related research literature studies at home and abroad [18, 19]. In addition, the assessment of the bearing capacity of the foundation is important in practice [20, 21], so the effective evaluation of the bearing capacity of an existing engineering pile group is also a great significant prerequisite to ensure its safe bearing capacity.

In this paper, the bearing capacity for the existing engineering pile group during the dynamic top-down construction for layer adding is studied as the research object. First, the linear eigenvalue method is used to research the influences of the lateral restraint on bearing capacity for the existing engineering pile group. Then, the nonlinear extreme point stability method with initial imperfection is used to study the amplification range of the corresponding calculation length. Finally, the reference suggestion for the value of the column pile stability coefficient in the existing code is proposed, which provides safety guarantee for the practical engineering application.

2. Connection Structure and Bearing Calculation Theory

2.1. Vertical Support Underpinning the Connection Structure. Taking a high-rise building with the existing pile foundation as an example, a new basement is built by using the technology of the top-down construction for layer adding. Figure 1 shows the scene photo of the existing high-rise building which was built in 1997 with 13 floors above the ground and 1 floor underground. Figure 2 shows the pile plan of the existing engineering piles.

The vertical support section diagram of the basement layer adding during the top-down construction is shown in Figure 3. A vertical underpinning structure supported by two or more piles is arranged at the bottom of each frame column for vertical load conversion. When the existing engineering pile group under the column is supported by a multiple pile group (such as two piles, three piles, and four piles), the principle of equivalent section bending stiffness can be adopted to simplify the analysis into a two-pile support in plane for unified consideration and to simplify calculation workload.

For the existing high-rise buildings, if only the vertical underpinning structure system is used to resist the horizontal action such as the wind load, its lateral stiffness will be very difficult to meet the requirements of the relevant code. In this case, by using the horizontal supporting beams of the foundation pit bracing structures (such as the diaphragm wall, the row pile), the upper main building and the vertical underpinning system can be well laterally restrained. Therefore, the bearing capacity calculation of the existing engineering pile group can be analyzed under the condition of no lateral displacement.

2.2. Dynamic Construction Sequence of the Top-Down Construction for Layer Adding

Step 1. Construction of the surrounding enclosure structures, including diaphragm walls and row piles, etc.

Step 2. The horizontal support beams are constructed and connected with the basement roof and the transfer-bearing platform of the existing building to form a vertical support system without lateral displacement.

Step 3. Excavation of the earthwork and construction of the temporary steel bracings between existing engineering piles, so that the engineering pile group is integrated as a whole.

Step 4. After excavation to the elevation of the foundation, the static pressure anchor pile for compensating load is constructed, and then, the foundation slab is poured.

Step 5. Construction of the basement exterior wall, frame column, and other vertical load-bearing components, and then the design strength should be reached.
Step 6. The temporary steel bracings between piles should be removed, and then, the existing pile groups within the basement floor height range are cut off.

2.3. Bearing Capacity Calculation. The upper and lower parts of the existing pile group are restrained by the lateral restraint of the constructed floor structure and the unexcavated foundation soil, respectively. The bearing capacity of the existing engineering pile group changes with the dynamic change of the lateral restraint. Therefore, the determination of the calculation length and the bearing capacity should be analyzed according to different working conditions, and the value should be designed according to the most unfavorable working condition [22–24].

According to Euler’s formula, the critical stability load $P_{ct}$ of the slender rod is as follows:

$$P_{ct} = \frac{\pi^2 E_{sc} I_{sc}}{(u L_{sc})^2}$$

(1)

where $E_{sc} I_{sc}$ is the bending stiffness of the section; $L_{sc}$ is the effective length; and $u$ is the effective length coefficient.

The corresponding effective length coefficient $u$ is as follows:

$$u = \frac{\pi}{L_{sc}} \sqrt{\frac{E_{sc} I_{sc}}{P_{ct}}}$$

(2)

First, the stability load $P_{ct}$ of the existing engineering pile group is obtained by the finite element method, and then the value of effective length coefficient $u$ is obtained by substituting $P_{ct}$ into (2). For the linear eigenvalue analysis, the stability load is the critical load corresponding to the first buckling mode. For the nonlinear stability analysis, the stability load is the extreme point of the load-displacement curve. Due to the influence of initial imperfections, the actual nonlinear bearing capacity of the existing engineering pile groups is generally smaller than the ideal critical axial compression stability load in a linear eigenvalue analysis.

3. Finite Element Analysis Model

Considering the need to ensure the bearing performance of the upper structure in the excavation stage of the top-down construction for layer adding, the lateral restraint of the transfer bearing platform is provided by the horizontal support beam and the enclosure structure, which is the condition without lateral displacement. The eigenvalue stability method is applied to study the bearing capacity for existing engineering piles group under the dynamic top-down excavation.

3.1. Calculation Model and Simplification. Taking the second underground layer constructed by the top-down construction for layer adding as an example, the bearing platform of each frame column is simplified as a two-pile bearing platform. Considering the dynamic excavation of the foundation soil to the base elevation, the critical stability load and the effective length coefficient of the existing engineering pile group are analyzed.
The two-pile bearing platform is supported by bored piles with the diameter \( D \), the concrete grade C30, and the elastic modulus \( E_c = 30 \text{ MPa} \), which are imbedded in the rock layer (fixed support) at the bottom, and hinged or rigidly connected to the bearing platform. The two-pile bearing platform can be considered as an equivalent concrete beam supported by a one-way hinge at both ends, with the concrete grade C30, the rectangular section \( 2D \times 1.5 \text{ m} \), the elastic modulus \( E_{c0} \) = 30 GPa, and the length \( L_{b0} \) = 5D.

The temporary steel bracings are hinged or rigidly connected to the concrete piles at both ends, with the section H200 \( \times 100 \times 6 \times 8 \text{ mm} \) and the material Q235. When the excavation reaches the base, the depth of the concrete pile into the soil is \( h \), and the pile calculation part length \( L_c \) is 5.0 m. There is a vertical concentrated force \( P \) on the top center of the two-pile bearing platform, with the value \( P = 2N_0 = 2f.A_2 = 8086 \text{ kN} \). The calculation model is shown in Figure 4.

### 3.2. Horizontal Resistance of the Foundation Soil

The horizontal elastic resistance is subjected to the embedded part of the concrete pile by the foundation soil [25–27]. The \( m \) method is used here for assuming that the horizontal elastic resistances \( q(x) \) of the foundation soil increases linearly with the depth of the soil \( x \), which is shown by the following equation:

\[
q(x) = \begin{cases} 
  m(h - x)yb_0, & (x \in [0, h]) \\
  0, & (x \in [h, L]) 
\end{cases}
\]

where \( b_0 \) is the calculated width of the concrete piles (m), with the value in this case of \( b_0 = 0.9 \times (1.5D + 0.5); D \) is the pile diameter (m); \( m \) is the horizontal resistance coefficient of the foundation soils (MPa/m²); \( x \) is the embedded depth of the concrete piles into the soil layer (m); \( y \) is the horizontal lateral displacement of the concrete piles (m); and \( L \) is the total length of the concrete piles (m).

### 3.3. Finite Element Model

Based on the ABAQUS, the horizontal elastic resistance is simulated by the unidirectional grounding spring element with an interval of 1.0 m. The spring stiffness \( K_a \) is determined according to \( (3) \), as shown by the following equation:

\[
K_a = \begin{cases} 
  m(h - x)b_0 \times 0.5, & (x \in [0, h]), \\
  0, & (x \in [h, L]) 
\end{cases}
\]

where \( x_a \) is the x-coordinate of each spring (m) and \( K_a \) is the elastic stiffness of each spring (N/m). The finite element analysis model is shown in Figure 5.

### 4. Influencing Factors Analysis for the Bearing Capacity

The main influence factors on the bearing capacity for existing engineering pile group include the horizontal elastic resistance of the foundation soils, the diameter of piles body, and the bending stiffness of the temporary steel bracings between piles.

As for the linear eigenvalue stability method, by considering the existing engineering pile group without initial imperfection, the load corresponding to the first buckling mode is obtained as the critical ultimate load. This method is simple and fast in calculation and convenient for a large number of parametric analysis, which is adopted for analysis in this section.

#### 4.1. Influence of the Horizontal Resistance of Foundation Soils

Taking the pile diameter \( D = 600 \text{ mm} \) as an example, the influence of the general non-rock foundation soil \( m = 0.1\sim6.0 \text{ MPa/m²} \) on the bearing capacity for existing engineering piles group is studied, excluding the effect of the temporary steel bracing.

The curves of the critical load factor \( \alpha \) and the effective length coefficient \( u \) with the \( m \) value are shown in Figures 6 and 7, respectively, corresponding to the critical loading \( P_{cr} = \alpha P \) and the effective length \( L_{cr} = uL_c \). It is shown in Figure 6 that the critical loading factor \( \alpha \) increases with the increase of the \( m \) value and is shown in Figure 7 that the effective length coefficient \( u \) decreases with the increase of the \( m \) value. When \( m > 2.0 \text{ MPa/m²} \), the change ranges of the \( \alpha \) value and the \( u \) value becomes slow. This is due to that, when the \( m \) value increases, the lateral restraint for foundation soils increases, then the lateral displacement of the lower end of the calculation part (base elevation) decreases, then the corresponding effective length decreases and the bearing capacity increases.

It turns out that the bearing capacity for the case of the rigid pile top is higher than that of the case of the hinged pile top, and the effective length coefficient is opposite. Taking \( m = 1.0 \text{ MPa/m²} \) as an example, the \( \alpha \) value of the case of the hinged pile top and the case of the rigid pile top are 8.439 and 13.573, respectively, and the bearing capacity of the latter is about 1.61 times of the former. Therefore, strengthening the level of the rigid connection for the pile top is an effective way to improve the bearing capacity during the top-down construction.

#### 4.2. Influence of the Pile Diameter

Taking the horizontal resistance coefficient \( m = 1.0 \text{ MPa/m²} \) as an example, the pile diameter \( D \) is taken as the commonly used range of 600 mm~900 mm for analysis. The curves of the load factor \( \alpha \) and the effective length coefficient \( u \) with the pile body diameter \( D \) are shown in Figures 8 and 9. The results show that the influence of the pile body diameter \( D \) are great on the bearing capacity and the effective length for both two cases of the pile top connection. Moreover, the \( \alpha \) value increases with the increases of the \( D \) value, while the \( u \) value decreases with the increases of the \( D \) value.

#### 4.3. Influence of the Bending Stiffness of Temporary Steel Bracing

The influence of the temporary steel bracing on the stability of the concrete pile is caused by the lateral restraint on the middle position of the pile, which is due to the bending stiffness of the steel bracing. Taking \( m = 1.0 \text{ MPa/m²} \) and \( D = 600 \text{ mm} \) as an example, the hinged joint and the
Figure 4: Analysis model and simplification. (a) Geometric model. (b) Mechanical model.

Figure 5: Finite element analysis model.
rigid joint are considered respectively for the connection of the steel bracing and the concrete pile. The linear stiffness ratio of the steel bracing to the concrete pile is shown by the following equation:

\[ K_1 = \frac{i_{b1}}{i_c} = \frac{(E_{b1}I_{b1})/(E_c I_c)}{i_c} \]  

(5)

where \( E_{b1} I_{b1} \) and \( E_c I_c \) are the bending stiffness of the equivalent concrete beam and the pile calculation part, respectively; \( L_{b1} \) and \( L_c \) are the lengths of them. In the case of this section, the value of \( K_1 \) in this paper is 0.257, and the range of \( K_1 = 0.05 \sim 5.0 \) is selected for analysis.

The curves of the critical load factor \( \alpha \) and the effective length coefficient \( u \) with the \( K_1 \) value are shown in Figures 10 and 11, respectively, with the case of the hinged pile top (Case 1) and the case of rigid pile top (Case 2). It can be seen that for both cases, when the connecting ends of the steel bracing are hinged, its influence on the stability of the existing engineering pile group is very small, which is equivalent to the condition without support. When the connecting ends of the steel bracing are rigid, the critical load factor \( \alpha \) increases with the increases of the \( K_1 \) value, while the effective length coefficient \( u \) decreases with the increases of the \( K_1 \) value. Moreover, when \( K_1 > 2.0 \), the
changes of $\alpha$ and $u$ tend to be gentle. This is due to the fact that the rotational restraint of the steel bracing on the middle position of the pile group increases with the increase of the $K_1$ value, which causes the rise of the reverse bending point at the lower end of the calculation part, the reduction of the effective length, and the increase of the bearing performance.

When the connecting ends of the steel bracing are rigid, the bearing capacity of the Case 2 is higher than that of the Case 1. Taking $K_1 = 1.0$ as an example, the bearing capacity ratio of the Case 1 and the Case 1 is 1.388 (rigid bracing) and 1.599 (hinged bracing). Therefore, adding more rigid steel bracings and strengthening the level of the rigid connections are effective ways to improve the bearing capacity during the top-down construction.

5. Nonlinear Bearing Capacity with Initial Imperfections

The nonlinear stability analysis belongs to the problem of the extreme point instability. Taking the second underground layer constructed by the top-down construction for layer adding in Section 3 as an example, the initial geometric imperfections are applied in the analysis to obtain the full stable equilibrium path of the existing engineering pile group.

5.1. Types of the Initial Geometric Imperfection. There are two types for the initial geometric imperfection: the consistent mode imperfection and the lateral displacement imperfection. For the former, the first buckling mode of the engineering pile group is taken as its initial imperfection, and the maximum deformation is the imperfection amplitude. For the latter, the deformation of the engineering piles group caused by the lateral displacement of the bearing platform is taken as the initial imperfection, and the end lateral displacement is the imperfection amplitude. The imperfection amplitude is generally selected as $1/1000 \sim 1/200$ of the pile group length.

5.2. Influence of the Consistence Mode Imperfection. Taking $m = 1.0$ MPa/m$^2$ and $D = 600$ mm as an example, the hinged pile top (Case 1) and the rigid pile top (Case 2) are considered for analysis, excluding the effect of the temporary steel bracing. The total length of the pile group is 32m, and the corresponding imperfection amplitudes $w = 35$ mm, 100 mm and 160 mm are selected.

The curves of the critical load factor $\alpha$ and the effective length coefficient $u$ with the axial displacement $v$ of the top node for different imperfection amplitudes are shown in Figures 12 and 13, respectively. For both the case 1 and the case 2, there are obvious extreme points in each load-displacement curve. The nonlinear stability load with the initial imperfection is slightly smaller than the linear eigenvalue stability load, and the corresponding nonlinear effective length is slightly larger than the linear effective length, in which the former is more in line with the practical engineering state. The larger the amplitude of the initial imperfection, the smaller the critical stability load.

The $\alpha$ value and the $u$ value of different imperfection amplitudes are shown in Table 1, where $\alpha_0$ and $u_0$ is the load factor and the effective length coefficient of the linear eigenvalue buckling analysis, respectively. It can be seen that when the range of the imperfection amplitude is 35 mm~160 mm, the nonlinear effective length coefficients of the case 1 and the case 2 are about 1.04~1.18 times and 1.04~1.15 times of the corresponding linear effective length coefficients, respectively. In the practical engineering design, it is suggested that the effective length of the existing engineering pile group should be multiplied by the amplification factor...
\[ \beta = 1.20 \] to consider the influence of the initial imperfections.

5.3. Influence of the Lateral Displacement Imperfection. The model parameters and the imperfection amplitude are described in Section 5.2, and the initial imperfection form is selected as the lateral displacement imperfection form of the foundation pit. The curves of the critical load factor \( \alpha \) and the effective length coefficient \( u \) with the axial displacement \( v \) of the top node for different imperfection amplitudes are shown in Figures 14 and 15, respectively. For both the case 1 and the case 2, the effect of the lateral displacement imperfection on the values of \( \alpha \) and \( u \) is much smaller than that of the consistent mode imperfection, which means that the first buckling mode deformation is the most unfavorable initial imperfection. The critical stability load decreases with the increase of the imperfection amplitude.

It turns out from Table 1 that when the ranges of the imperfection amplitude is 35 mm–160 mm, the nonlinear effective length coefficients of the case 1 and the case 2 are about 1.01–1.06 times and 1.02–1.05 times of the corresponding linear effective length coefficients, respectively. In the practical engineering design, it is suggested that the
Table 1: Values of $\alpha$ and $u$ for different imperfection amplitudes.

| Case imperfection amplitude (mm) | Case 1 | Case 2 |
|----------------------------------|--------|--------|
|                                  | 100    | 160    | 100    | 160    |
| Linear                           |        |        |        |        |
| $\alpha_0$                       | 8.439  | 13.573 |        |        |
| $u_0$                            | 1.486  | 1.72   |        |        |
| $v$ (m)                          | 0.17   | 0.26   |        |        |
| Consistent mode imperfection     |        |        |        |        |
| $\alpha$                         | 7.859  | 6.864  | 12.463 | 11.297 | 10.291 |
| $u$                              | 1.540  | 1.648  | 1.754  | 1.223  | 1.284  | 1.346  |
| $u/u_0$                          | 1.04~1.18 | 1.04~1.15 |        |        |
| Nonlinear                        |        |        |        |        |
| Lateral displacement imperfection |        |        |        |        |
| $\alpha$                         | 8.283  | 7.915  | 7.585  | 13.150 | 12.682 | 12.386 |
| $u$                              | 1.500  | 1.534  | 1.567  | 1.190  | 1.212  | 1.227  |
| $u/u_0$                          | 1.01~1.06 | 1.02~1.05 |        |        |

Figure 14: Load factor of the lateral displacement imperfection. (a) Case 1. (b) Case 2.

Figure 15: Effective length coefficient of the lateral displacement imperfection. (a) Case 1. (b) Case 2.
effective length of the existing engineering pile group should be multiplied by the amplification factor $\beta = 1.10$ to consider the influence of the initial imperfections.

### 6. Bearing Capacity considering the Buckling Effect

The present solution of the nonlinear effective length coefficient $u_c$ of the existing engineering pile group should be taken as follows:

$$u_c = \beta u.$$  \hspace{1cm} (6)

where $u$ is the effective length coefficient obtained from the linear eigenvalue stability analysis in Section 4 and $\beta$ is the amplification factor of the effective length coefficient in Section 5 considering the nonlinear effect of the initial imperfection. For the sake of safety, it can be taken as $\beta = 1.20$.

Considering the compression buckling effect of the high-rise cap pile, and according to the items 6.3.10 $\sim$ 6.3.11 in technical specification for building foundation excavation engineering constructed by the top-down method (DB33/T1112-2015) [28] and the items 5.8.2 $\sim$ 5.8.4 in technical code for building pile foundation (JGJ 94-2008) [29], the bearing capacity $N_{u}$ of the pile body considering the compression buckling effect in this paper can be obtained, as shown by the following equation:

$$N_u = \phi \psi c f_c A_c,$$  \hspace{1cm} (7)

where $\phi$ is the compression buckling coefficient and $\psi$ is the pile-forming technology taken as the median value 0.75.

The comparison between the present solution and the code solution is shown in Table 2, where $l_o$ is the actual length of the pile calculation part and $\alpha$ is the horizontal deformation coefficient of the pile. Considering the need of safety for the engineering design, the effective length of the single pile considering compression buckling is considered as the unfavorable working condition of the hinged pile top.

It can be seen from Table 2 that the present solution is about 1.10 $\sim$ 1.30 times of the code solution for the bearing performance of the existing engineering piles group, and the code solution is relatively safe. In practical engineering design, the code solution of the bearing capacity can be relaxed by 10% $\sim$ 30% according to the actual situation and can also be considered as the design margin.

### 7. Conclusion

In this paper, the linear eigenvalue stability analysis and the nonlinear stability analysis are applied to study the effective length coefficient and obtain the bearing capacity with the buckling effect. The conclusions are as follows:

(1) The lateral restraint is the determining factor affecting the bearing capacity of the existing engineering piles group. Moreover, the critical stability load and the effective length coefficient of the engineering pile group can be obtained by the finite element method.

(2) For both the hanged pile top and the rigid pile top, the critical stability load factors $\alpha$ increase with the increases of the horizontal resistance coefficients $m$, while the corresponding effective length coefficient $u$ decrease. When $m > 2.0$ MPa/m², the change ranges of the $\alpha$ value and the $u$ value becomes gentle.

(3) The critical stability load factor $\alpha$ increases with the increase of both piles group diameter $D$ and bracing bending stiffness $K_b$. Moreover, when $K_b > 2.0$, the changes of $\alpha$ and $u$ tend to be gentle. The hinged temporary steel bracing has no effect on the stability of the existing engineering pile group.

(4) The nonlinear stability load with the initial imperfection is slightly smaller than the linear eigenvalue stability load, and the larger the amplitude of the initial imperfection, the smaller the nonlinear critical stability load. In practical engineering, the effective length coefficients of the existing engineering pile group with the consistent mode imperfection and the lateral displacement imperfection should be enlarged by 1.20 and 1.10 times, respectively.

(5) The present solution is about 1.10 $\sim$ 1.30 times of the code solution on the bearing capacity for existing engineering piles group, and the code solution is relatively safe. In practical engineering design, the code solution of the bearing capacity can be relaxed by 10% $\sim$ 30% according to the actual situation and can also be considered as the design margin.
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.

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