Numerical study on the effect of supporting pile calculation methods in pile-anchor support system

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Article

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Abstract

In numerical analysis, the pile calculation method is a key factor in foundation pit calculation results. However, the mechanism of different pile calculation methods in the foundation pit has remained unclear. To evaluate performances of different pile calculation methods in pile-anchor support system, four pile calculation methods (equivalent solid elements without interface (Mode1), equivalent solid elements with interface (Mode2), one-dimensional structural elements (Mode3), and two-dimensional plate elements (Mode4)) were used in a typical pile-anchor pit based on FLAC3D. Results show that: there is a great influence on the computational results for different pile calculation methods: (1) solid pile elements (Mode1 and Mode2), compared with structural pile elements (Mode3 and Mode4), displacements near pile are smaller, with the increase of excavation depth, stress concentration appeared in solid pile elements. (2) Solid support pile, with interface (Mode2) compared with without interface (Mode1), the former of that was less displacement, whereas more stress concentration around the pile, which leads special points stress paths around the solid pile to become irregular. (3) Regarding structural pile elements (Mode3 and Mode4), it suggested both modeling approaches have similar effect. For this foundation pit, Mode2 is more suitable. If Mode3 and Mode4 are used for this pit design, it will obviously exaggerate the actual displacement and overestimate the construction cost. The results of this paper can provide a deeper understanding of pile calculation in pile-anchor foundation pits numerical analysis.

1. Introduction

With the rapid development of urbanization, the construction land resources are becoming tighter and tighter, the use of underground space has become the key direction of research, large shopping malls, subway stations, and other buildings in the construction process cannot be separated from the problem of foundation pit engineering. so the excavation and support of the foundation pit is a hot issue for research. The pile-anchor support structure method is the tie-back method that the retaining structure is braced through anchors embedded into the neighboring grounds. Such a method allows the foundation pit to be obstacle-free thereby ensuring sufficient working space for constructions, hence, is widely used in foundation pit projects.

Research on pile-anchor support structure mainly includes the empirical formula method and physical model method, although both methods provide guidance for pit construction, however, the former oversimplifies the actual site and is often too conservative in predicting the calculation results, for the latter, due to physical model is usually a scaled-down model, the selection of similar materials and physical parameters has a great impact on the results and is economically expensive. Recently, with the continuous development of computer technology, numerical simulation methods have become an important way to deal with the problem of foundation pits, and are widely used in the design of foundation pit engineering.

Pile structure is the main component of the pile-anchor support structure, in numerical analysis, pile support structure plays an important role and all relevant results are influenced by it. Regarding the pile...
calculation methods, considerable results have been reported. In general, the pile calculation methods could be divided into four types: the first type was equivalent to linear elastic solid elements\textsuperscript{13,14,15}, the equivalent stiffness criterion was selected to transfer piles to diaphragm wall \textsuperscript{16,17}, as shown in Eq. (1), which D is the diameter of pile, m; t is the pile spacing of piles, m; and h is the thickness of retaining wall, m.

\[
\frac{th^3}{12} = \frac{\pi D^4}{64}
\]

The second type is the same as the first type, with an additional pile-soil interface element. While the third type is one-dimensional structural elements\textsuperscript{18,19}, and the fourth type was equivalent to two-dimensional plate elements\textsuperscript{14,20,21}, the thickness of the wall is equated according to Eq. (1). Recently, Shao, et al.\textsuperscript{15} considered the piles as solid elements and the influence of the pile-soil interface to analyze the deformation of the foundation pit. Li, et al.\textsuperscript{14} carried out the study of pit deformation by considering the outer piles of the pile structure as equivalent solid elements and the inner piles as equivalent two-dimensional plate elements. Wang, et al.\textsuperscript{19} considered the pile structure as a one-dimensional structural element to study an ultra-deep pile-anchored support foundation pit, etc. However, researchers usually use one or two types of these pile simulation methods to conduct relevant studies\textsuperscript{13,14,15,20,21}. The mechanism of different pile simulation methods in foundation pit has remained unclear.

In this paper, four pile calculation methods were evaluated based on a typical pile-anchor foundation profile by FLAC3D. The differential results generated by the four pile simulation methods were investigated from the perspective of soil displacement and stress, special points stress path, and plastic zone. And mechanism differences of different pile simulation methods were analyzed. The results of this paper can provide a deeper understanding of the pile calculation in pile-anchor foundation pits numerical analysis.

### 2. Three-dimensional Finite-difference Model

#### 2.1. Typical foundation pit profile

A pile-anchor foundation pit site in Shenzhen, China was shown in Fig. 1, the typical profile was shown in Fig. 2. The soil parameters of the profile were divided into four layers from top to bottom: artificial fill, layer thickness was around 1m; granite residual soil, layer thickness was 20.5m; fully weathered granite, layer thickness was near 4.8m; strongly weathered granite. This pit profile is representative, the support structures involved in this profile include slope spray mixing, piles, crown beam, waist beams, and anchors. The construction process information is shown in Table 1.
<table>
<thead>
<tr>
<th>Stage</th>
<th>Construction sequences</th>
</tr>
</thead>
</table>
| 1     | initial $K_0$ stress field  
pile installation |
| 2     | 1st excavations to the depth of GL.-1.0m,  
slope surface spray |
| 3     | 2nd excavations to the depth of GL.-5.0m, the first layer of anchors installation at GL.-4.5m |
| 4     | 3rd excavations to the depth of GL.-8.5m, the second layer of anchors installation at GL.-8.0m |
| 5     | 4th excavations to the depth of GL.-12.0m, the third layer of anchors installation at GL.-11.5m |
| 6     | 5th excavations to the depth of GL.-14.9m |

### 2.2. Soil layer and support structure parameters

(1) The built-in plastic hardening (PH) model in FLAC3D was selected for soil constitutive models, which is realistic for describing excavation of foundation pits. Soil parameters refer to the region experience of Qin, et al. The values of soil parameters were shown in Table 2.
Table 2
Input soil parameters in PH model

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>elements</th>
<th>Soil1 (0-1m)</th>
<th>Soil2 (1-21.5m)</th>
<th>Soil3 (21.5-26.3)</th>
<th>Soil4 (26.3-50m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{50,ref}$</td>
<td>Primary loading stiffness (reference)</td>
<td>kN/m²</td>
<td>6,500</td>
<td>2,210</td>
<td>72,000</td>
<td>100,000</td>
</tr>
<tr>
<td>$E_{oed,ref}$</td>
<td>Oedometric stiffness (reference)</td>
<td>kN/m²</td>
<td>6,500</td>
<td>2,210</td>
<td>72,000</td>
<td>100,000</td>
</tr>
<tr>
<td>$E_{ur,ref}$</td>
<td>Un/reloading stiffness (reference)</td>
<td>kN/m²</td>
<td>19,500</td>
<td>6,630</td>
<td>216,000</td>
<td>300,000</td>
</tr>
<tr>
<td>$v_{ur}$</td>
<td>Poisson's ratio</td>
<td></td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td>0.24</td>
</tr>
<tr>
<td>Cohesion</td>
<td>kN/m²</td>
<td></td>
<td>8</td>
<td>25</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>$\varphi$</td>
<td>Friction angle</td>
<td></td>
<td>12</td>
<td>22</td>
<td>25</td>
<td>35</td>
</tr>
<tr>
<td>$\psi$</td>
<td>Dilatancy angle</td>
<td></td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>m</td>
<td>Stress dependency index</td>
<td></td>
<td>0.5</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Elements weight</td>
<td>kN/m³</td>
<td>19</td>
<td>19.5</td>
<td>19.5</td>
<td>20.5</td>
</tr>
<tr>
<td>$K_0$</td>
<td>Earth pressure coefficient at rest</td>
<td></td>
<td>$1 - \sin \varphi$</td>
<td>$1 - \sin \varphi$</td>
<td>$1 - \sin \varphi$</td>
<td>$1 - \sin \varphi$</td>
</tr>
<tr>
<td>$p_{ref}$</td>
<td>Reference pressure</td>
<td>kN/m²</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

(2) The support structure parameters are shown in Table 3. Slope spray mixing was considered by shell elements, the anchor was considered by cable elements, piles were considered by equivalent solid elements, pile elements, and liner elements, respectively, based on different ways of pile calculation, crown beams were considered by solid elements and beam elements, and waist beams were divided into two cases of considering waist beams and not considering waist beams, which was considered by beam elements, more details are in the 2.3 section.
2.3. Model assumptions

Following assumptions were made for the efficient model operation.

1. In the process of simulation excavation, the site was flat after each excavation, and each over-excavation was 0.5 m. Anchor construction and application of prestressing were completed instantly, and there was no loss of prestress.

2. Since the time for excavation is relatively short, hence, the consolidation analysis was ignored\(^8\).

2.4. Model element mesh and boundary conditions

Considering geometric spacing of the pile anchors and the grid quality, the model was 7.8 m lengths in the direction of extension outside the vertical profile and was divided into 12 equally spaced grids with a spacing of 0.65 m each. The position of the first two anchor points along the outer extension direction of the vertical profile are indicated by black triangles (Fig.3), the third layer of anchors is a black rectangular box. The horizontal motion was constrained at the lateral boundary, while both horizontal and vertical motions were constrained at the bottom boundary of the model (Fig.3). In addition, the distance between the pile and the outer boundary of the mesh was ensured to be larger than three times the final excavation depth to minimize the boundary effect\(^8\). When the soil is dug out, the NULL model is
applied\textsuperscript{22}. The excavation and support stages of the foundation pit are in accordance with the actual construction stages, which are shown in Table 1.

2.5. Pile calculation methods

According to the previous research results of scholars, four types of pile calculation methods were considered: equivalent solid elements without the interface (Fig.4a), equivalent solid elements with the interface (Fig.4b), one-dimensional structural elements (Fig.4c), and two-dimensional plate elements (Fig.4d), respectively.

(1) Equivalent solid elements without interface (Mode1)

As for Mode1 (Fig.4a), according to Eq.(1), based on the geometric parameters of the piles shown in Fig.2, the equivalent thickness $h$ was 0.77m, which was simulated using solid elements. No interface elements were used at the soil-pile interface, which means soil-pile interface displacement is continuous\textsuperscript{26}. A linear-elastic constitutive model was adopted for the solid elements. And the crown beam was considered by solid elements and given the same stiffness parameter values as the solid elements. Without considering the setting of the waist beam, the anchor head nodes were in rigid contact at the intersection with the solid elements.

(2) Equivalent solid elements with interface (Mode2)

In Mode2 (Fig.4b), the pile, crown beam, and anchors were considered as same as the Mode1, but with the addition of soil-pile interface element, the shear behavior of the pile-soil interface obeyed the Mohr-Coulomb criterion. Since there was no interface between the pile and soil in Mode1, they were considered to be in good contact\textsuperscript{21}. For comparison with Mode1, the interface shear strength parameter of Mode2 was set as the soil layer parameter using the control variable method, hence, the interface friction angle was taken as 22° and the cohesion was 25 kPa, that was the shear strength index of the second layer soil. For at pit, the normal and tangential stiffness of the interface were recommended as Eq.(2)\textsuperscript{22}.

$$K_n = K_s = 10 \max \left[ \frac{K + 4 / 3G}{\Delta Z_{\text{min}}} \right]$$

Where $K_n$ and $K_s$ are normal and tangential stiffness of interface element, $K$ and $G$ are the bulk modulus and shear modulus of the soil layer, respectively, which can be calculated by transforming Young's modulus and Poisson's ratio; represents the dimension of the grid near the interface element along the direction of the vertical interface element. According to the relevant parameters, the normal and tangential stiffness were taken 246 MPa. The anchors have moved half the distance of wall thickness...
toward to wall inside in its X-directional horizontal position compared to Mode1, in order to avoid the anchors from affecting the sliding of the interface element on this side of the pit.

(3) One-dimensional structural elements (Mode3)

In Mode3 (Fig. 4c), the pile structure was simulated with a one-dimensional structural element, namely, pile elements. To prevent soil flow between piles, the reinforcement area within the pile diameter of 1.2 m was considered (Green part of Fig. 4c), the reinforcement area was simulated with a linear-elastic constitutive model, its deformation parameter was three times the deformation modulus of the surrounding soil, the crown beam and waist beam were simulated with beam elements. Beam, anchor, waist beam, and pile structure elements were in rigid contact at the intersection.

(4) Two-dimensional plate elements (Mode4)

For Mode4 (Fig. 4d), the pile was equivalent to plate elements according to the equal stiffness method and was simulated with two-dimensional structural elements, namely, embedded liner elements, which consider the pile-soil coupling spring effect. The liner thickness and the pile-soil interface parameters were set in the same way as the Mode2. The crown beam was simulated by beam elements, waist beam was ignored. The crown beam, anchor, and liner elements were in rigid contact at the intersection.

3. Result

3.1. Foundation pit displacement

Since the pit extended along with the vertical profile with a length of 7.8 m, considering the pile anchor setting and geometric symmetry, the vertical profile at 3.9 m in the Y direction was selected to extract the displacement for analysis.

3.1.1. Comparison analysis of wall displacement for four simulation methods

Figures 5(a-e) shows pile wall lateral displacement under different excavation stages. It can be seen that: When the excavation depth was relatively small, the difference among the results of the four modes was not significant. (Fig. 5a), when excavation depth is deeper, only the lateral displacement of Mode2 increases slowly, while the other three modes increase rapidly (Fig. 5b, Fig. 5c). At the end of the excavation, the Mode1 wall top lateral displacement reached 1.75 times that for Mode2, meanwhile, the displacement distribution curves of Mode3 and Mode4 were relatively similar, and their lateral displacement in wall top was 2 times that for Mode2. (Fig. 5e).

3.1.2. Comparison analysis of ground surface displacement for four simulation methods
Figures 6(a-e) shows ground surface displacement for four simulation methods under different excavation stages. Except for Mode2, in different excavation stages, the ground settlement tank is basically the same in the other three simulation methods. In addition, the ground surface settlement distributions of four simulation methods under various excavation stages showed similar characteristics to the lateral displacement, this means that for the same pile calculation method, there is a good agreement between the lateral displacement and the ground surface settlement performance, the detail of those values would be discussed later in Section4.1.

3.1.3. Comparison analysis of measurement result and numerical result

Figure 5(c), Fig. 5(e), Fig. 6(e) show that the result of Mode2 is much closer to the monitor value.

3.2. Soil stress

3.2.1. Comparison analysis of soil special point stress paths for four simulation methods

Soil special points stress paths might provide useful explanations for soil stress changes caused by pit excavation. In this paper, five representative special points were selected near the wall, the analysis of stress paths under different pile simulation methods were carried out (the locations of points A, B, C, D, and E are shown in Fig. 7(a), where the light blue figure was a schematic diagram of the pit at the end of excavation stage). Points A and B were located near the bottom of the pile (17.6 m from the top of the pile), where point A was located at the front side of the pile, while point B was located at the backside of the pile, point C was located at the backside of the pile near the upper part of the pile (2.3 m from the top of the pile), point D was located at the backside of the pile in the middle and upper part of the pile (5.8 m from the top of the pile), and point E was located at the backside of the pile at the location of the end excavation (13.9 m from the top of the pile). Respectively, the Arabic numbers 1, 2, 3...6 represent the initial stress field stage, 1st excavation, 2nd excavation ... 5th excavation. Since all the five special points were located within the second layer of soil, the five special points shared the same $K_f$ and $K_0$ lines.

Figure 7 shows the comparison of stress paths at special points for four modes under various excavation stages. In the initial stress field stage, as for these five special points, the s-t values are all located at the $K_0$ line for the four different simulations, which indirectly indicated the accuracy of the model simulation in the initial stress field. As for Mode1, Mode3 and Mode4: points A and B, as excavation depth deepens, because point A was at the bottom of the excavation area, the stress unloading level was much more intense compared to point B, resulting in a smaller mean stress at point A and a greater shift of its coordinates towards the left (Fig. 7a, c, d), at the same time, because the horizontal stress unloading level at point A was less than the vertical, its deviator stress would become larger. As for points C, D, and E, which were located at the backside of the pile, their stress paths were all unloaded strongly horizontally,
that was, the minimum principal stress decreased continuously. From (Fig. 7a, c, d), it can be found that as excavation depth deepened, the coordinates of these three points moved to the left and moved steadily from the initial $K_0$ line to the $K_f$ line.

For Mode2, it can be found from Fig. 7(b) that the trend of stress paths at these five special points during the first four excavation stages was similar to the other three modes. At the end of the excavation, the mean stress at point A decreased dramatically compared to that of the other three modes, from 250 kPa to nearly 160 kPa, and the decrease in shear stress caused it to drop below the $K_0$ line, while the mean stress at point B increased slightly and moved below the $K_0$ line. meanwhile, the mean stress at points C and D increased. The mean stress at point E increased intensely, from 175 kPa in the previous stage to 250 kPa, the reasons for the details would be discussed later in Section 4.2.

3.2.2. Comparison analysis of soil plastic zone for four simulation methods

Plastic zone statistics are generally used as a validation tool to evaluate slope and tunnel stability. It was introduced to the comparison evaluation for four pile calculation methods, considering two yield properties, tension and shear yield. There are two ways of calculating yield zone in FLAC3D: the first way is that when more than 50% of the volume of a soil zone has yielded, the soil zone is considered to yield; the second way is that when any volume of a soil zone has yielded, the soil zone is considered to yield. The first way was adopted for calculation. Where the shear yield percent was defined as the ratio of the volume of soil that generated shear yield to the volume of all soil zone for a certain excavation stage and considering the cumulative value of the previous excavation stage. The definition of tensile yield percent was similar to that of shear yield percent.

As the excavation depth deepens, the magnitudes and percent of shear yield volume resulting from the four modes were that: Mode3 > Mode1 > Mode4 > Mode2 (Fig. 8). While the magnitudes and percent of tension yield volume resulting from the four modes were that: Mode4 > Mode3 > Mode1 > Mode2 (Fig. 9). The plastic zone in Mode2 is minimum.

4. Discussion

4.1. Comparison of simulation results with empirical methods

Clough and O'Rourke indicated that $\delta_{ehm}/H_e$ ($\delta_{ehm}$ represents the maximum lateral displacements of the wall in the final excavation stage, $H_e$ represents final excavation depth) was around 0.2%, with the upper bound of 0.5%. The results of the four simulation methods in the final excavation stage (Fig. 10) were close to 0.4% for Mode3 and Mode4, while Mode2 was below about 0.2%, Mode1 was among them. In this paper, we extended the study to each excavation stage, ($\delta_{hm}$ represents the maximum lateral displacements under various excavation stages, is the depths of the excavation corresponding to the
excavation stage). It can be found that the results of Mode1, Mode3, Mode4 were nearly consistent before the final excavation stage, around 0.2%, while Mode2 increased from 0.0–0.2% with the deepening of the excavation depths.

Similarly, Clough and O’Rourke\textsuperscript{33} indicated that $\delta_{evm}/H_e$ ($\delta_{evm}$ represents the maximum ground surface displacements in the final excavation stage) was around 0.15%, with an upper bound of 0.5%. The results of the four simulation methods in the final excavation stage (Fig. 11) were close to 0.3% for Mode3 and Mode4, while Mode2 was below about 0.2%, Mode1 was also among them. Similarly, extending its study to each excavation stage, ($\delta_{vem}$ represents the maximum ground surface settlements under various excavation stages). As the excavation deepened, results of Mode1, Mode3, and Mode4 were nearly consistent before the final excavation stage, increased from 0.05–0.15%, while Mode2 increased from 0.0–0.15%.

As shown in Fig. 12, the maximum lateral displacement of the same pile calculation method corresponded well with the ground settlements\textsuperscript{34}. And the relationship between maximum lateral displacement and ground settlement generated by four modes can be described by the same linear equation.

4.2. Control mechanism of soil stress by four pile calculation method

Figure 13 shows stress tensor and the stress diagram in XX direction for four pile simulation methods at the end of the excavation, and the cross-section was selected at the same position as section 3.1.

The piles in Mode1 (Fig. 13a) and Mode2 (Fig. 13b) were simulated by equivalent solid elements. Due to large differences in stiffness parameters between the equivalent solid elements and the soil (Table 2, Table 3), the equivalent solid elements deformation was very small, stress discharge of the equivalent solid elements was less than the nearby soil, so the solid elements stress tensor changed drastically, which was also confirmed by the fact that there was no excessive change in stress of the reinforcement area in Mode3 (Fig. 13c). Besides, the stress concentration was observed at two special locations in Mode1, respectively, at the head of the first layer anchor and the second layer anchor, since the anchor head of Mode1 was in rigid contact with equivalent solid elements, the displacement of both was synergistic at that location. The reason why stress concentration occurred only at the two locations was due to the rapid increase of the anchor axial force in these two layers during this excavation stage (Fig. 14).

The maximum horizontal stresses of Mode1, Mode3, and Mode4 were the same, all of them were less than 500kPa. As for Mode2, the stress concentration pattern showed block distribution. The maximum horizontal compressive stresses reached 720 kPa, which was 1.44 times the other three modes, located at the side behind the wall at the bottom of the excavation face. While point E was located in the area of intense stress concentration, the stress concentration was more intense, therefore the point E stress path was different from the other three modes (Fig. 7(b)). By checking the interface element (Fig. 15), it is
found that both normal stresses of the interface element and shear stress were maximum in this part of the area, therefore, it is suggested that the interface element caused the stress concentration in this area.

As for Mode3 and Mode4, due to anchor heads were only rigidly connected to the structural elements, without the existence of equivalent solid elements with a large difference in stiffness with the adjacent soil, hence no stress concentration occurred, and the stress distribution and stress tensor distribution were essentially same for both modes. Also, since the magnitudes of the stress tensor near the pile side in Mode3 and Mode4 were smaller than that in Mode1 and Mode2, the level of stress unloading of Mode3 and Mode4 was more drastic, resulting in greater deformation, which could also be evidenced by the larger lateral displacement and ground displacement of Mode3 and Mode4 (Fig. 5e, Fig. 6e).

5. Conclusion

In this paper, four pile calculation methods were evaluated based on a typical pile-anchor foundation profile by FLAC3D. The difference results derived from four pile calculation methods were investigated from the perspective of soil displacement and stress, special points stress path, and plastic zone. The main conclusions can be drawn as follows:

(1) Solid pile elements, compared with structural pile elements, displacements near pile are smaller, with the increase of excavation depth, stress concentration appeared in solid pile elements.

(2) As for solid support pile, with interface compared with without interface, the former of that are less displacement, whereas more stress concertation around the pile, which leads special points stress paths around the solid pile to become irregular.

(3) Regarding structural pile elements, one-dimensional structural elements compared to two-dimensional plate elements, displacements and stresses are the same, which suggests both modeling approaches have the same effect.

To sum up, in 3D pile-anchor deep excavation numerical model, the results difference obtained from different pile calculation methods cannot be ignored, of which, the solid pile may cause less displacement and more stress concertation, for this foundation pit, in terms of displacement, solid pile elements with interface element is more suitable. If Mode3 and Mode4 are used for this pit design, it will obviously exaggerate the actual displacement and overestimate the construction cost. The results of this paper can provide a deeper understanding of the pile calculation in pile-anchor foundation pits numerical analysis.

Declarations

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**Author contributions**

Xiaoyang Liu: Conceptualization, Writing-Original draft preparation, Software. Zhiguo Chen: Resources. Changming Wang: Supervision, Writing - review & editing, Funding acquisition.

**Data availability statement**

The data that support the finds of this study are available from the corresponding author upon reasonable request.

**Additional Information**

Competing Interests Statement

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

**References**


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Comparison of simulation results and empirical methods with maximum settlement under various excavation stages
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\[ y = 0.704x - 1.902 \]

\[ R^2 = 0.98 \]
Figure 13

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Figure 15

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