Integral Seismic Deformation Method: A Pseudo-static Method for Seismic Design of Underground Structures

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Abstract: The traditional seismic deformation method is among the most popular pseudo-static methods for seismic design of underground structures by considering soil-structure interaction through the layout of foundation springs. It can be mechanically proved that the traditional method has a relatively rigorous theory except the discrete foundation springs. A new kind of pseudo-static method, called the integral seismic deformation method, is proposed in this paper by establishing the finite element models of hollow ground and integral soil-and-structure. Equivalent seismic loads of the proposed method are composed of three parts: shear stress of free field, inertia force of underground structure and equivalent nodal forces acquired by the static analysis of the hollow ground model. In order to verify the efficiency of the proposed method, a series of numerical calculations of Daikai subway station are conducted by the two pseudo-static methods and the dynamic time-history analysis, respectively. Comparison of the calculation results shows that the proposed method achieves high precision and extensive applicability. It is also convenient and time-saving to operate. Therefore, the integral seismic deformation method can be widely used for seismic design of underground structures.

1. Introduction
Recently, the development and utilization of underground space have become one of the most important ways to deal with population growth, land resources shortage and environmental issues in many countries. As we know, subways, railways, highways, material storage and sewage transport have become important facilities of modern society[1]. It is expected that the cost of excavating underground space will decrease within the next thirty years while the cost of constructing superstructures will rise rapidly[2]. Developing underground space will become more economical than constructing superstructures in the future, simultaneously having many benefits for ecology and environment[3]. Underground structures had been assumed to have better seismic capacity than superstructures by the 1990s[4], but severe damages have been reported in some recent earthquakes, including the 1995 Hyogoken-Nanbu (Japan) earthquake[5], the 1999 Chi-Chi (Taiwan) earthquake[6], the 2003 Bam (Iran) earthquake[7], and the 2011 Tohoku (Japan) earthquake[8]. Because of the importance for life save and economy, the safety of underground structures is drawing more and more attention. Owing to the surrounding soil, the dynamic response of underground structures is obviously different from superstructures, as they are subjected to ground distortions, rather than inertial loads[1]. Hence, the seismic response of underground structures has become a research focus and the seismic design is necessary especially in earthquake-active areas.

There are basically two categories of approaches for seismic analysis of underground structures, namely the dynamic time-history analysis and the pseudo-static methods[9]. In dynamic time-history analysis, the computational model includes underground structure and a finite, bounded soil zone...
adjacent to the structure, and the particular ground motion is imported from transmitting boundaries to the discretized domain to simulate wave propagation[10]. This approach has high precision, but it is too complicated and time-consuming to be employed in the general seismic design. While the pseudo-static method is a simplified method aiming to convert the dynamic problem into equivalent static problem, which makes it more popular in practical engineering designs.

Various researches on the pseudo-static methods have been conducted so far. St John and Zahrah[11] put forward the free-field deformation method for an elastic tunnel by ignoring the interaction between the underground structure and surrounding soil. Wang[12] proposed the free-field racking method to consider the soil-structure interaction by introducing the racking coefficient. Penzien[13] presented an analytical procedure for evaluating the racking deformation of rectangular and circular tunnel linings caused by soil-structure interaction during a seismic event based on Wang’s method. Kiyomiya[14] proposed the ground deformation method by connecting the underground structure to the ambient ground through shear and normal springs, and imposed the enforced displacements to the calculation model. Gil[15] put forward a simplified method for the analysis of square cross-section buried structures under P and S waves by imposing the free-field displacements on the boundaries of the influenced zone of the structure. Liu[16] proposed a pushover analysis method for underground structures, and applied this method to seismic damage assessments. Zou[17] modified Liu’s pushover method with more reasonable load distribution, and compared the results with shaking table tests and dynamic time-history analyses.

Among the existing pseudo-static methods, the seismic deformation method, also called response deformation method[18–20], has become a widespread approach for seismic design of underground structures and piles in several design codes in different countries, such as China and Japan. The seismic deformation method for transverse seismic analysis of underground structures establishes the structure-foundation springs computational model, imposing the inertial force and shear stress on the structure and relative displacements on the free ends of the springs. Compared with other pseudo-static methods, the seismic deformation method is theoretically rigorous and takes into account the effect of soil-structure interaction and inertial force. However, the computational error of the seismic deformation method is significant sometimes, because the coefficients of foundation springs are determined approximately and the discrete springs cannot reflect the internal action of ground soil itself either.

The principal objective of this paper is to develop an improved pseudo-static method for the seismic design of underground structures based on the seismic deformation method. The subsequent presentation is organized as follow. First, the mechanical analysis of the underground structures subjected to seismic loads is conducted, which is also the basic theory of the seismic deformation method. Accordingly, the disadvantages of the seismic deformation method are indicated. Second, an improved pseudo-static method is proposed by modifying the seismic deformation method with replacing the ground springs. Finally, the calculation results of the Daikai subway station subjected to seismic loads are compared with those of dynamic time-history analyses in order to verify the efficiency of the proposed method.

2. Mechanical analysis of the underground structure subjected to earthquake

In general, a rectangular underground structure subjected to seismic load \( \{\ddot{u}_s\} \) from bedrock is shown in Fig.1, in which the interface is the contact surface between the structure and the surrounding soil. The stress states of the hollow ground soil and the underground structure are respectively shown in Fig.2 (a) and (b).
Figure 1. Schematic chart of underground structure subjected to seismic load

Figure 2. Stress states of the hollow ground and the underground structure

where \( \{F\} \) and \(-\{F\}\) are the interaction forces on the interface, \( \{u\} \) and \( \{u_I\} \) are, respectively, the displacement vectors of the underground structure and the interface.

The basic motion equation of the underground structure is given by Eq.(1).

\[
\begin{bmatrix}
    m_I & 0 \\
    0 & m_S
\end{bmatrix}
\begin{bmatrix}
    \dot{u}_I \\
    \dot{u}_S
\end{bmatrix}
+ \begin{bmatrix}
    k_I & k_{IS} \\
    k_{SI} & k_S
\end{bmatrix}
\begin{bmatrix}
    u_I \\
    u_S
\end{bmatrix}
= \begin{bmatrix}
    -F \\
    0
\end{bmatrix}
\tag{1}
\]

Subsequently, the hollow ground soil subjected to \( \{u_I\} \) and \( \{u\} \) can be decomposed into two stress states, as shown in Fig.3.

Figure 3. Stress states decomposition of the hollow ground soil

where \( \{u_I\} \) and \( \{u_{I2}\} \) are the displacement solutions of the hollow ground soil subjected to \( \{u_e\} \) and \( \{F\} \), respectively.

The summation of \( \{u_I\} \) and \( \{u_{I2}\} \) is equal to the displacement vector on the interface \( \{u_I\} \), as given in Eq.(2).
The relationship between \( \{u_{i1}\} \) and \( \{F\} \) is given by Eq.(3).

\[
[K] \{u_{i2}\} = \{F\}
\]

where \([K]\) is the ground impedance determined by the hollow ground soil.

Substitution of Eq.(2) into Eq.(3) gives Eq.(4).

\[
\{F\} = -[K]\{u_{i1}\} + [K]\{u_{i}\}
\]

Substitution of Eq.(4) into Eq.(1) gives Eq.(5).

\[
\begin{bmatrix}
  k_{II} + K k_{II} & k_{II} \\
  k_{II} & k_{II}
\end{bmatrix}
\begin{bmatrix}
  \{u_{i}\} \\
  \{u_{s}\}
\end{bmatrix}
= \begin{bmatrix}
  m_{II} & 0 \\
  0 & m_{II}
\end{bmatrix}
\begin{bmatrix}
  \{\ddot{u}_{i}\} \\
  \{\ddot{u}_{s}\}
\end{bmatrix}
\]

It can be seen that Eq.(5) is the original equation of the seismic deformation method. In order to obtain the seismic response of the underground structure \( \{u_{i1}\} \), the displacement of the hollow ground soil under earthquakes is needed. Nishida[21] suggested the “technique using displacement of hollow ground” to analyze the displacement relationship between the free field and the hollow ground soil. Schematic charts of mutual relationships among the displacements of free field and the hollow ground soil are shown in Fig.4. The free-field displacement on the interface nodes is given as \( \{u_{0}\} \), and the shear stress of free field acting on the interface (framed by dashed lines in Fig.4(a)) is assumed to be \( \{F_{0}\} \). Moreover, the displacement vector of the hollow model is given as \( \{u_{1}\} \) subjected to the shear stress \( \{F_{1}\} \). Hence, the summation of \( \{u_{0}\} \) and \( \{u_{1}\} \) makes \( \{u_{i}\} \) as given in Eq.(6), and the relationship between \( \{u_{1}\} \) and \( \{F_{1}\} \) is given in Eq.(7).

\[
\{u_{i}\} = \{u_{0}\} + \{u_{1}\}
\]

\[
[K]\{u_{i}\} = \{F_{0}\}
\]

Substitution of Eq.(6) into Eq.(7) yields:

\[
[K]\{u_{i}\} = \{F_{0}\} + [K]\{u_{0}\}
\]

Substitution of Eq.(8) into Eq.(5) yields:

\[
\begin{bmatrix}
  k_{II} + K k_{II} & k_{II} \\
  k_{II} & k_{II}
\end{bmatrix}
\begin{bmatrix}
  \{u_{i}\} \\
  \{u_{s}\}
\end{bmatrix}
= \begin{bmatrix}
  m_{II} & 0 \\
  0 & m_{II}
\end{bmatrix}
\begin{bmatrix}
  \{\ddot{u}_{i}\} \\
  \{\ddot{u}_{s}\}
\end{bmatrix}
\]

Substitution of Eq.(9) into Eq.(5) yields:

\[
\begin{bmatrix}
  k_{II} + K k_{II} & k_{II} \\
  k_{II} & k_{II}
\end{bmatrix}
\begin{bmatrix}
  \{u_{i}\} \\
  \{u_{s}\}
\end{bmatrix}
= \begin{bmatrix}
  m_{II} & 0 \\
  0 & m_{II}
\end{bmatrix}
\begin{bmatrix}
  \{\ddot{u}_{i}\} \\
  \{\ddot{u}_{s}\}
\end{bmatrix}
\]

It can be seen that Eq.(9) is the original equation of the seismic deformation method. In order to obtain the seismic response of the underground structure \( \{u_{i1}\} \), the displacement of the hollow ground soil under earthquakes is needed. Nishida[21] suggested the “technique using displacement of hollow ground” to analyze the displacement relationship between the free field and the hollow ground soil. Schematic charts of mutual relationships among the displacements of free field and the hollow ground soil are shown in Fig.4. The free-field displacement on the interface nodes is given as \( \{u_{0}\} \), and the shear stress of free field acting on the interface (framed by dashed lines in Fig.4(a)) is assumed to be \( \{F_{0}\} \). Moreover, the displacement vector of the hollow model is given as \( \{u_{1}\} \) subjected to the shear stress \( \{F_{1}\} \). Hence, the summation of \( \{u_{0}\} \) and \( \{u_{1}\} \) makes \( \{u_{i}\} \) as given in Eq.(6), and the relationship between \( \{u_{1}\} \) and \( \{F_{1}\} \) is given in Eq.(7).

\[
\{u_{i}\} = \{u_{0}\} + \{u_{1}\}
\]

\[
[K]\{u_{i}\} = \{F_{0}\}
\]

Substitution of Eq.(6) into Eq.(7) yields:

\[
[K]\{u_{i}\} = \{F_{0}\} + [K]\{u_{0}\}
\]

Substitution of Eq.(8) into Eq.(5) yields:

\[
\begin{bmatrix}
  k_{II} + K k_{II} & k_{II} \\
  k_{II} & k_{II}
\end{bmatrix}
\begin{bmatrix}
  \{u_{i}\} \\
  \{u_{s}\}
\end{bmatrix}
= \begin{bmatrix}
  m_{II} & 0 \\
  0 & m_{II}
\end{bmatrix}
\begin{bmatrix}
  \{\ddot{u}_{i}\} \\
  \{\ddot{u}_{s}\}
\end{bmatrix}
\]

Eq.(9) is the fundamental equation of the seismic deformation method. The left part of the equation is the response of underground structure considering the impedance of the hollow ground soil. Meanwhile, the right contents are, respectively, the free-field shear stress acting on the interface, the reaction forces acting on the interface to make the hollow ground reach the free-field displacement, and the inertial force of the underground structure. Based on the Eq.(9), the seismic deformation method could be proposed. Fig.5 shows the computational model of the seismic deformation method, in which the normal and shear ground springs are introduced to take into account the impedance of the
hollow ground soil. Fig.6 shows the determinations of the spring coefficients by means of the finite element method.

\[ k_N = \frac{P_N}{\delta_N} \]
\[ k_S = \frac{P_S}{\delta_S} \]

where \( k_N \) and \( k_S \) are the coefficients of normal springs and shear springs, \( P_N \) and \( P_S \) are the pressure and shear stress imposed on the side of the hollow, \( \delta_N \) and \( \delta_S \) are the average normal strain and average shear strain, respectively.

3. Methodology of the integral seismic deformation method

In terms of the derivation process of Eq.(9), the basic theory of the seismic deformation method is rigorous. Nevertheless, the coefficient determinations of the ground springs are approximate, which may lead to large computational errors in some cases. In order to avoid this kind of computational error, an improved pseudo-static method is proposed in this section.

As mentioned before, the introduction of the ground springs is for the sake of the impedance of the hollow ground soil. In Eq.(9), the stiffness matrix \([K]\) reflects the impedance of the hollow ground soil, and \([K]\{u_0\}\) is the reaction forces acting on the interface to make the hollow ground have the free-field displacement. Since the hollow ground soil plays an important role in the seismic analysis of underground structure, it would be a better way to build an integral model of the underground structure and the surrounding soil, instead of building a series of discrete ground springs.

In the traditional seismic deformation method, the free-field displacement is imposed on the end of the ground springs to apply the load \([K]\{u_0\}\) in Eq.(9). While in the new method, the hollow ground model is built, on which the free-field displacement is imposed, as shown in Fig.7. Through static
analysis, the reaction forces acting on the interface are taken as the equivalent nodal forces, which are exactly $[K]\{u_0\}$ in Eq.(9).

Figure 7. Calculation of the equivalent nodal forces through hollow ground model

Fig.8 shows the computational model of the new pseudo-static method proposed in this paper. The loads imposed on the model consist of three parts, which are, respectively, the shear stress around the structure, the inertial force of the structure and the equivalent nodal forces on the interface obtained from the static analysis of the hollow ground model.

Figure 8. Computational model of the integral seismic deformation method

The implementation procedure of the integral seismic deformation method is presented as follow.

(1) Seismic analysis of the free field. Obtain the displacement, acceleration and shear stress of the free field corresponding to the interface position. In order to simulate the nonlinear response of soil, the equivalent-linear approach can be utilized[22, 23]. The subsequent soil properties of computational models are determined by the final iteration of the equivalent-linear analysis. Some existing programs are available for equivalent-linear seismic response analysis of the free field such as SHAKE91[24], EERA[25] and ProShake[26].

(2) Calculation of the equivalent nodal forces. Build the computational model of hollow ground and impose the free-field displacement acquired by step (1) on the interface as shown in Fig.7. Obtain the equivalent nodal forces on the interface through the static analysis of the hollow ground soil.

(3) Seismic analysis of the underground structure. Build the integral soil-structure interaction model as shown in Fig.8, and conduct static analysis of the integral model by imposing the free-field shear stress, the inertial force and the equivalent nodal forces. The inertial force could be approximately calculated by multiplying the mass of structure by the free-field acceleration.

Fig.9 shows the whole solving procedure of the proposed method.
4. Verification of the integral seismic deformation method

4.1. Analysis model and ground motions

In order to verify the computation accuracy of the proposed method, the typical cross-section of Daikai subway station is chosen as the research object, which suffered severe damage in 1995 Hyogoken-Nanbu earthquake of Japan[27]. The model dimension of Daikai station is shown in Fig.10. The thickness of the overburden soil is 4.8 m. The column area is 0.4×1 m² and the column space is 3.5 m. The material of structural beams and columns are assumed to be elastic, with the elastic young modulus of 30 GPa, the density of 2.49 g/cm³ and the poisson ratio of 0.15. The soil medium is assumed to be homogeneous and isotropic, with the original properties of various layers listed in Table 1. To approximately simulate the nonlinear response of soil medium, the shear modulus reduction curve and the damping ratio curve are established in Fig.11[22]. The controlling sections are respectively marked as A, B, C, and D in Fig.10. Three earthquake records (Loma Prieta 1989, Kobe 1995 and El Centro 1940) shown in Fig.12-14, respectively, are used as the ground motions with the peak of acceleration 0.2g.
Figure 11. Strain-dependent shear modulus and damping of ground soil

Table 1. Soil properties of each layer

| Soil layer | Depth (m) | Density (g/cm³) | Shear wave velocity (m/s) | Poisson ratio |
|------------|-----------|-----------------|---------------------------|---------------|
| 1          | 0~1.0     | 1.9             | 140                       | 0.33          |
| 2          | 1.0~5.1   | 1.9             | 140                       | 0.32          |
| 3          | 5.1~8.3   | 1.9             | 170                       | 0.32          |
| 4          | 8.3~11.4  | 1.9             | 190                       | 0.40          |
| 5          | 11.4~17.2 | 1.9             | 240                       | 0.30          |
| 6          | 17.2~39.2 | 2.0             | 330                       | 0.26          |
| 7          | >39.2     | 2.1             | 500                       | 0.30          |

Figure 12. Acceleration record of Loma Prieta

Figure 13. Acceleration record of Kobe
4.2 Calculation results
We performed the proposed integral seismic deformation method (ISDM) on the Daikai station and compared the results with dynamic analysis method (DAM). The results of the controlling sections are shown in Table 2. It can be seen that the results of ISDM are extremely close to those of DAM, with the computational errors all less than 5%.

| Ground motion | Method | Moment of A (kN·m) | Moment of B (kN·m) | Moment of C (kN·m) | Moment of D (kN·m) | Relative horizontal displacement of A-B (mm) | Relative horizontal displacement of C-D (mm) |
|---------------|--------|---------------------|---------------------|---------------------|---------------------|-------------------------------------------|-------------------------------------------|
| Loma Prieta   | ISDM   | 364.829             | 452.920             | 52.966              | 53.352              | 6.428                                     | 6.486                                     |
|               | DAM    | 364.307             | 450.305             | 53.734              | 53.418              | 6.364                                     | 6.422                                     |
| Kobe          | ISDM   | 387.694             | 478.076             | 56.423              | 56.902              | 6.805                                     | 6.865                                     |
|               | DAM    | 400.833             | 502.289             | 59.159              | 59.595              | 7.069                                     | 7.132                                     |
| El Centro     | ISDM   | 395.270             | 491.640             | 58.016              | 58.473              | 7.049                                     | 7.110                                     |
|               | DAM    | 395.910             | 488.785             | 58.528              | 58.528              | 7.020                                     | 7.081                                     |

4.3 Applicability of the proposed method
In order to verify the applicability of the proposed method, we performed this method on the Daikai station in a series of computing conditions. Specifically, the variation parameters were, respectively, thickness of overburden soil, strength of underground structure, and stiffness of ground soil. The structure and soil are assumed to be elastic, and the ground motion imposed is Loma Prieta record. The results of controlling sections are shown in Table 3~5, compared with those of the seismic deformation method (SDM) and DAM.

| Thickness of overburden soil | Method | Moment of A (kN·m) | Moment of B (kN·m) | Moment of C (kN·m) | Moment of D (kN·m) | Relative horizontal displacement of A-B (mm) | Relative horizontal displacement of C-D (mm) |
|-----------------------------|--------|---------------------|---------------------|---------------------|---------------------|-------------------------------------------|-------------------------------------------|
| 2m                          | SDM    | 198.184             | 258.102             | 32.154              | 33.542              | 3.784                                     | 3.811                                     |
|                             | ISDM   | 191.979             | 270.220             | 28.654              | 29.910              | 3.376                                     | 3.413                                     |
|                             | DAM    | 199.991             | 283.942             | 30.525              | 31.746              | 3.512                                     | 3.550                                     |
| 4.8m                        | SDM    | 273.425             | 328.962             | 41.067              | 41.791              | 4.846                                     | 4.885                                     |
|                             | ISDM   | 284.656             | 371.627             | 36.83               | 37.823              | 4.403                                     | 4.464                                     |
|                             | DAM    | 290.681             | 379.185             | 38.671              | 38.997              | 4.429                                     | 4.492                                     |
| 11.5m                       | SDM    | 312.758             | 330.651             | 37.301              | 36.913              | 4.941                                     | 4.993                                     |
|                             | ISDM   | 356.767             | 416.377             | 34.290              | 34.688              | 4.610                                     | 4.706                                     |
|                             | DAM    | 373.579             | 446.505             | 37.095              | 37.227              | 4.763                                     | 4.868                                     |
| 17.2m                       | SDM    | 297.151             | 316.798             | 35.210              | 34.239              | 3.766                                     | 3.826                                     |
### Table 4. Calculation results of different structure strengths

| Structure stiffness | Method | Moment of A (kN·m) | Moment of B (kN·m) | Moment of C (kN·m) | Moment of D (kN·m) | Relative horizontal displacement of A-B (mm) | Relative horizontal displacement of C-D (mm) |
|---------------------|--------|--------------------|--------------------|--------------------|--------------------|------------------------------------------|------------------------------------------|
| Half                | SDM    | 178.500            | 216.798            | 23.186             | 24.183             | 5.613                                    | 5.678                                    |
|                     | ISDM   | 195.828            | 268.129            | 20.671             | 22.223             | 5.127                                    | 5.245                                    |
|                     | DAM    | 206.515            | 283.799            | 22.874             | 23.834             | 5.367                                    | 5.495                                    |
| Original            | SDM    | 273.425            | 328.962            | 41.067             | 41.791             | 4.846                                    | 4.885                                    |
|                     | ISDM   | 284.656            | 371.627            | 36.830             | 37.823             | 4.403                                    | 4.464                                    |
|                     | DAM    | 290.681            | 379.185            | 38.671             | 38.997             | 4.429                                    | 4.492                                    |
| Twice               | SDM    | 398.748            | 472.235            | 65.621             | 66.561             | 3.863                                    | 3.887                                    |
|                     | ISDM   | 397.949            | 494.766            | 59.486             | 60.345             | 3.534                                    | 3.565                                    |
|                     | DAM    | 404.738            | 501.719            | 61.227             | 61.557             | 3.547                                    | 3.578                                    |

It can be seen from Table 3~5 that the relative errors of bending moments between ISDM and DAM are all less than 10%, and the relative errors of relative horizontal displacements are even smaller. By comparison, the relative computing errors between SDM and DAM are larger. The moments of lateral wall and the relative horizontal displacements calculated by SDM are smaller than those of DAM, but the moments of middle column are bigger, which means that a bigger rigid body rotation of lateral walls occurs in the calculation results of SDM. Moreover, with the thickness of overburden soil or soil stiffness decreasing, or structure strength increasing, the difference between SDM and DAM is reducing. In other words, the relative errors between SDM and DAM decrease along with the reduction of the stiffness ratio of soil and structure. The relative errors in SDM are brought by the ground springs. First, foundation springs are independent elements that cannot accurately simulate the internal action of ground soil itself. With the ratio of soil to structure increasing, the constraint of soil on structure enhances, making up for the deficiency of SDM. Second, the coefficients of ground springs are determined so approximately that the rigidity of the middle domain on each side of the hollow is overestimated but the rigidity of the corners is underestimated.

Compared with the traditional seismic deformation method, the integral seismic deformation method has higher precision in different situations. In conclusion, the proposed method has a good applicability, and it can be widely used in seismic designs of actual projects.
5. Conclusions
Based on the basic theory of the seismic deformation method, we proposed a new pseudo-static method for seismic analysis of underground structures, called the integral seismic deformation method. By comparing the results of the traditional method and the new one with dynamic time-history analysis, some conclusions can be drawn as follow.

(1) The computation errors brought by the ground springs established in the traditional method cannot be ignored. Not only the coefficients of ground springs are determined approximately, but also the discrete springs cannot actually simulate the internal action of ground soil, especially when the stiffness ratio of soil and structure is high.

(2) The integral seismic deformation method is theoretically rigorous. By establishing the hollow ground model, the equivalent nodal forces are calculated instead of the coefficients of ground springs. Besides, the soil-structure interaction model is established for seismic analysis, containing three parts of input loads, namely shear stress of free field, the inertial force of structure, and the equivalent nodal forces on the contact face.

(3) The proposed method has high accuracy and good applicability. The applicability of the integral seismic deformation method is verified by comparing the calculation results of Daikai subway station with dynamic analysis method through changing the parameters, namely, the thickness of overburden soil, the strength of underground structure, and the stiffness of ground soil. In addition, the proposed method is time saving. By building the integral soil-structure computing model, there is no need to determine the coefficients of foundation springs.

The integral seismic deformation method proposed in this paper is effective and accurate in estimating the seismic response of underground structures. It is also time-saving and convenient to operate. Therefore, it can be widely used in seismic assessments and designs of practical underground structures. Moreover, further work is required to study on the accurate effect of inertia force, especially for the large-space and large-span underground structures. A generalized software program is also needed to make this new method programmatically used.

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