## Article

# Load Calculation Method for Deep-Buried Layered Soft Rock Tunnel Based on Back-Analysis of Structural Deformation 

Lixin Zhang © Lijun Chen *, Jianxun Chen, Yanbin Luo, Huijie Guo, Yang Zhai and Pengkun Wang

School of Highway, Chang'an University, Xi'an 710064, China; 2018021022@chd.edu.cn (L.Z.); chenjx1969@chd.edu.cn (J.C.); lyb@chd.edu.cn (Y.L.); 17782794650@163.com (H.G.); zhaiyang@chd.edu.cn (Y.Z.); wpk0918@163.com (P.W.)

* Correspondence: chenlijun@chd.edu.cn; Tel.: +86-18700870229

Citation: Zhang, L.; Chen, L.; Chen, J.; Luo, Y.; Guo, H.; Zhai, Y.; Wang, P. Load Calculation Method for Deep-Buried Layered Soft Rock Tunnel Based on Back-Analysis of Structural Deformation. Symmetry 2024, 16, 383. https://doi.org/ 10.3390/sym16040383

Academic Editor: Theodore E. Simos

Received: 5 March 2024
Revised: 21 March 2024
Accepted: 22 March 2024
Published: 23 March 2024


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#### Abstract

After the excavation and unloading of deep-buried soft rock tunnels, support structures often experience deformation-related disasters such as concrete cracking, steel frame bending and twisting, and primary support instability under different forms of load. Accurately calculating the load borne by the primary support structure is the key to ensuring design rationality and construction safety. Especially in layered soft surrounding rock formations, the magnitude and distribution of the loads are different from those of conventional rock and soil masses, resulting in limited applicability of existing load calculation methods to similar formations. Therefore, based on the measured deformation of the tunnel structure, while considering the different geometric forms of the primary support structure during partial excavation, this paper proposes a deformation-structure (D-S) load calculation method. By comparing the calculation results of this method and a large number of sample data for typical deep-buried layered soft rock tunnels, the reliability of the D-S load calculation method is verified. In addition, the variation law of the loads during the tunnel construction period is enunciated, and the magnitude and distribution of the loads acting on the primary support are clarified. The D-S load calculation method provides a theoretical basis for load calculation in deep-buried layered soft rock tunnels.


Keywords: tunnel engineering; layered soft rock; load calculation method; primary support; structural deformation

## 1. Introduction

After the excavation and support of a deep-buried layered soft rock tunnel, the initial stress of the surrounding rock is released and re-distributed, causing the loose and weak rock mass to squeeze inward and forming different forms of force on the support structure, namely, the surrounding rock pressure [1,2]. As a type of flexible support, the primary support is prone to deformation under this surrounding rock pressure. Coupled with the fact that the location and direction of the surrounding rock pressure under complex rock mass conditions are unclear, the deformed primary support structure contacts the surrounding rock and produces a reactive force, namely, elastic resistance [3]. The surrounding rock pressure and elastic resistance, together, constitute the load acting on the primary support structure [4,5]. Determining the magnitude and distribution of the load is the prerequisite for the structural design of the tunnel and also the fundamental guarantee of the safe construction and long-term operation of the tunnel.

Since the rise of the mining industry in the 19th century, many scholars have generally considered surrounding rock pressure as the load acting on support structures [6]. With the emergence of various complex geological conditions, the understanding of the loads acting on support structures in underground engineering is becoming increasingly mature. Research on rock pressure has continued to deepen. Following this, representative methods for calculating surrounding rock pressure include the loose media theory (M. M. Promojiyfakonov theory, Terzaghi theory, Xie Jiaxiao theory, and Bierbaumer theory) [7-12],
statistical regression estimation $[13,14]$, surrounding rock classification system (Q-system methods and RMR system formula) [15-17], and elastic-plastic theory (Kastner formula, Fenner formula, and Caquot formula) [18-20], as well as the finite element principle (numerical calculation methods) [21-23]. Surrounding rock pressure calculation methods based on the loose media theory consider the influence of joints and fissures, and assume the existence of cohesion within the soft rock mass, which partly conforms to the actual situation of soft surrounding rock. However, these methods ignore the influence of geological structures on the surrounding rock pressure. When calculating the natural equilibrium arch axis and analyzing the force of thin-layered units, they consider the surrounding rock as an isotropic medium without considering the unique stress transfer path due to the layered rock mass, resulting in a discrepancy between the pressure distribution and the actual force form of the primary support structure [24]. A large number of statistical data are the foundation of the statistical regression estimation method and the surrounding rock classification system. However, insufficient sample data are the weak link in calculating the surrounding rock pressure of special geological formations. The sample capacity in deepburied layered soft rock still needs to be supplemented to increase relevant parameters and optimize the calculation results. The rock pressure calculation method based on elasticplastic theory is based on the ideal elastic-plastic medium that is homogeneous, continuous, and isotropic [25-27]. However, the layered surrounding rock has obvious anisotropic characteristics. The surrounding rock pressure is asymmetric and non-uniformly distributed, which does not meet the assumption of this theory [28-30]. To sum up, for deep-buried layered soft rock, the above rock pressure calculation methods have deficiencies such as over-idealized assumptions, inconsistency between rock stress states and actual conditions, insufficient targeted sample cases and data, complex theoretical derivation processes, and difficulties in selecting some parameters. Their applicability needs to be further verified by specific engineering examples.

The elastic resistance provided by surrounding rock cannot be neglected in cases of bias pressure. Different degrees of structural deformation and cross-sectional shapes often lead to significantly different distribution patterns of elastic resistance. Normally, the Kommerall method and Бугаева method are used to calculate the elastic resistance of circular tunnels [31]. The elastic resistance distribution range of these methods is based on the ideal assumption. However, the stiffness of the lining structure, the coefficients of elastic resistance, the deformation characteristics of the structure, and the combined effect of external loads on elastic resistance are not considered [32-34], resulting in an overestimation of the calculation results. Especially in deep-buried layered soft rock strata, tunnel structures often exhibit biased pressure and asymmetrical deformation due to the anisotropy characteristics of the layered rock mass [35-38]. The actual distribution of elastic resistance is significantly different from the assumed range of these methods. In addition, the Бугаева method is used to calculate the elastic resistance of arch curved-wall tunnels [39]. The distribution range of elastic resistance is determined by judging the positive and negative displacement values at the boundary, which, to some extent, takes into account the influence of structural deformation. However, under special geological conditions, tunnel deformation is related to objective factors such as rock strength, bedding angle, groundwater, support measures, and construction disturbance, which are complex and varied [40-42]. When using this method to calculate elastic resistance, the distribution form of elastic resistance is difficult to coordinate with the non-uniform deformation law. A situation where the actual detachment zone still bears the resistance of the surrounding rock may occur. Therefore, under the influence of the structural deformation, the characteristics of the layered surrounding rock, and the shape of the excavation section, the elastic resistance calculation methods mentioned above cannot meet the engineering precision requirements of deep-buried layered soft rock tunnels.

In summary, it is a prerequisite to determine the load distribution and magnitude for calculating the internal forces of tunnel structures. Existing methods for calculating surrounding rock pressure and elastic resistance have certain limitations in their applicabil-
ity to deep-buried layered soft rock tunnels. Therefore, to avoid the complex mechanical transfer mechanism of the non-continuous medium in the plasticity theory and being limited by the special engineering sample in the statistical regression estimation, and to solve the difficulty in determining physical and mechanical parameters, it is necessary to seek a load calculation method for deep-buried layered soft rock tunnels with clear calculation principles, a simple calculation process, and targeted calculation results, in order to accurately reflect the load magnitude and distribution acting on the tunnel structure.

## 2. Methodology

Compared with the uncertainty of the loads that act on deep-buried soft rock tunnels, the deformation of the primary support has the advantage of relatively easy access to data. Furthermore, the deformation of the primary support is the comprehensive reaction of the structure under the surrounding rock pressure and elastic resistance after tunnel excavation and support. The load transmitted from the surrounding rock can be more accurately determined by structural deformation. Therefore, this paper proposes a deformationstructure (D-S) load calculation method based on measured deformation values, specifically for calculating the loads acting on a specific section of a deep-buried soft rock tunnel. This method considers the influence of the tunnel sectional excavation on the deformation and stress of the support structure by establishing the load combination calculation model (the surrounding rock pressure and elastic resistance) of each construction stage. In addition, the spring element in this model can characterize the effect of the elastic resistance, indirectly reflecting the anisotropic characteristics of the layered surrounding rock. By calculating the tension or compression of the spring element, the elastic resistance zone can be determined, avoiding calculation errors caused by improper assumptions and obtaining accurate load values and their distribution forms.

### 2.1. Assumptions

Based on the characteristics of the primary support and the layered surrounding rock mass, the following assumptions are made for the D-S load calculation model [43-45]: (1) The support effect of the longitudinal advance guide tube and grouting measures is not considered. The formation of cavities between the surrounding rock and the primary support caused by improper construction is not considered, that is, the surrounding rock remains closely attached to the primary support. (2) The primary support is approximately viewed as a linear elastic body, satisfying the plane section assumptions. (3) The longitudinal displacement of the tunnel is neglected, that is, the primary support of the tunnel is in a plane strain state. (4) The elastic resistance is proportional to the deformation of the primary support, which conforms to the local deformation theory. (5) The layered rock mass is considered to be a transversely isotropic quasi-continuous medium.

### 2.2. Principle

During the tunnel construction process, the geometric shape and boundary conditions of the structure will be affected by the support type, excavation and support sequence, and additional constraints. Therefore, it is necessary to consider the combination of these influencing factors when constructing the D-S load calculation model by using multiple types of elements. For the D-S load calculation model, the primary support structure is discretized into $n$ elastic beam elements with a unit length, and the connection points between each beam element are regarded as element nodes. Elastic beam or elastic rod elements are used to simulate components such as feet-lock pipes, anchor rods, or anchor cables that provide constraints and support forces for the primary support structure. Depending on their different bearing characteristics and connection forms, they are hinged or rigidly connected to the elastic beam element nodes. In addition, the elastic resistance of the surrounding rock acting on the primary support is decomposed into constraint forces along the normal and tangential directions of the rock layer according to the strata inclination, simulated using normal spring elements and tangential spring elements that
only bear axial force, respectively. Moreover, a combination element of a zero-thickness plate, vertical spring, and horizontal spring elements is used to simulate the base elastic support considering the horizontal, vertical, and rotational displacements of the primary support structure on the soft surrounding rock foundation. In summary, all of them are two-dimensional elements that exist simultaneously in both local coordinate systems (LCS) and global coordinate systems (GCS). In the LCS, the tensile direction of the elastic beam element is taken as the positive $\bar{x}$-axis direction. The normal spring tension direction is taken as the positive $\bar{x}$-axis direction. The $\bar{y}$-axes are positive when rotated counterclockwise $90^{\circ}$ from the $\bar{x}$-axis. In the GCS, the $x$-axis is horizontal to the right, and the $y$-axis is vertical in the positive direction. The D-S load calculation model is shown in Figure 1.


Figure 1. D-S load calculation model.
The element stiffness matrix in the LCS can be established by using the physical and mechanical parameters of the materials. Through applying the coordinate transformation formula, the element stiffness matrix in the GCS can be obtained, as shown in Equation (1).

$$
\begin{equation*}
\left[k_{(i, j)}\right]=\left[T_{(i, j)}\right]^{T}\left[\bar{k}_{(i, j)}\right]\left[T_{(i, j)}\right] \tag{1}
\end{equation*}
$$

where $\left[k_{(i, j)}\right]$ represents the stiffness matrix of different elements in the GCS, $\left[\bar{k}_{(i, j)}\right]$ represents the stiffness matrix of different elements in the LCS, and $\left[T_{(I, j)}\right]$ represents the coordinate transformation matrix of different elements. Different element types and support structure parts are represented by $i$ and $j$, respectively.

Under different construction stages, the actual support state of the support structure under complex constraints can be reflected by expanding and overlaying the stiffness matrix of each element into the corresponding position of the overall stiffness matrix. The representation of the total stiffness matrix is shown in Equation (2).

$$
\begin{equation*}
\left[K_{j}\right]=\sum_{i}\left[K_{(i, j)}\right] \tag{2}
\end{equation*}
$$

where $\left[K_{j}\right]$ represents the total stiffness matrix of the primary support under a certain construction stage. $\left[K_{(i, j)}\right]$ represents the total stiffness matrix of different elements under a certain construction stage.

Based on the deformation compatibility and static equilibrium conditions, the nodal displacements of each element connected at the same node are equal, and the loads acting on a node are balanced with the nodal forces of all elements acting on that node. A nodal displacement matrix ( $\left[\Delta_{j}\right]_{2 m \times 1}$ ) is formed by monitoring the displacement values of m $(m<n)$ key nodes. [ $\left.K_{j}\right]_{2 m \times 3 n}$ and $\left[\Delta_{j}\right]_{2 m \times 1}$ are brought into the equation of the primary support structure stiffness, as shown in Equation (3). The load of the key nodes ( $\left[P_{j}\right]_{2 n \times 1}$ )
can be solved. The load value at other nodes can be determined by linear interpolation according to their relative positions.

$$
\begin{equation*}
\left[P_{j}\right]=\left[K_{j}\right] \cdot\left[\Delta_{j}\right] \tag{3}
\end{equation*}
$$

The node displacement matrix of the primary support structure can be obtained by multiplying both sides of Equation (3) by the overall stiffness inverse matrix ( $\left[K_{j}\right]^{-1}$ ), as shown in Equation (4).

$$
\begin{equation*}
\left[\Delta_{j}\right]=\left[K_{j}\right]^{-1} \cdot\left[P_{j}\right] \tag{4}
\end{equation*}
$$

The node displacement matrix of the spring element in the LCS can be calculated by coordinate transformation. Based on the structural stiffness equation, the node load of the spring element can be obtained, as shown in Equation (5).

$$
\begin{equation*}
\left[\bar{F}_{(r, j)}\right]=\left[\bar{k}_{(r, j)}\right]^{T} \cdot\left[\bar{\Delta}_{(r, j)}\right] \tag{5}
\end{equation*}
$$

According to the principle of the elastic resistance mechanism, the existence of the spring element can be determined. If the normal node load at the spring element is under tension $\left(F_{(r, j)}>0\right)$, the spring element at that position should be cancelled. The above calculation steps are repeated until there are no spring elements under tension. Finally, the equivalent node load matrix of the primary support structure can be obtained.

## 3. Calculation

Taking the three-benching method commonly used in deep-buried layered soft rock tunnels as an example, the D-S load calculation model is established. The specific model construction and load calculation process is as follows. The primary support structure is discretized into $n-1$ elastic beam elements of unit-length, starting from the left arch foot of the upper bench. Encoding is performed along the arch axis clockwise and $n-1$ groups of elements and the corresponding $n$ nodes are formed. Spring elements are applied at all element nodes to represent the elastic resistance of the surrounding rock. Base elastic support elements are applied at the arch foot of the tunnel structure. The D-S load calculation model of the upper bench structure is shown in Figure 2.


Figure 2. The D-S load calculation model of the upper bench structure.

### 3.1. Establishment of Element Stiffness Matrix

According to the equilibrium conditions and the relationship between force and displacement in material mechanics, the stiffness matrix of the primary support in the LCS
can first be obtained. Then, by transforming the coordinate, the stiffness matrix of the primary support in the GCS is obtained, as shown in Equations (6) and (7).

$$
\begin{align*}
& {\left[k_{(s, u)}\right]=\left[\begin{array}{cccccc}
S_{1} & S_{2} & S_{3} & -S_{1} & -S_{2} & S_{3} \\
& S_{4} & S_{5} & -S_{2} & -S_{4} & S_{5} \\
& & 2 S_{6} & -S_{3} & -S_{5} & S_{6} \\
& & S_{1} & S_{2} & -S_{3} \\
& & & S_{4} & -S_{5} \\
& & & 2 S_{6}
\end{array}\right]}  \tag{6}\\
& \left\{\begin{array}{l}
S_{1}=\frac{12 E I \sin ^{2} \alpha_{i}}{l^{3}}+\frac{E A \cos ^{2} \alpha_{i}}{l} \\
S_{2}=\frac{E A \sin \alpha_{i} \cos \alpha_{i}}{l}-\frac{12 E I \sin \alpha_{i} \cos \alpha_{i}}{l^{3}} \\
S_{3}=-\frac{6 E I \sin \alpha_{i}}{l^{2}} \\
S_{4}=\frac{12 E I \cos ^{2} \alpha_{i}}{l^{3}}+\frac{E A \sin ^{2} \alpha_{i}}{l} \\
S_{5}=\frac{6 E I \cos _{i} \alpha_{i}}{l} \\
S_{6}=\frac{2 E I}{l}
\end{array}\right. \tag{7}
\end{align*}
$$

where $E, I, l$, and $A$ are the elastic modulus (MPa), moment of inertia $\left(\mathrm{m}^{4}\right)$, length (m), and cross-sectional area $\left(\mathrm{m}^{2}\right)$ of the beam element, respectively. $\alpha_{i}$ represents the angle of the beam element between the LCS and the GCS in different positions $\left({ }^{\circ}\right)$.

According to the bearing characteristics of different support components, the support components that can limit the structural angular displacement are regarded as elastic beam elements. The stiffness matrix of this type of support component is shown in Equation (6). Support components with a relatively small proportion of bending moments are regarded as elastic rod elements. The stiffness matrix of this type of support component in the GCS is shown in Equations (8) and (9).

$$
\begin{align*}
& {\left[k_{(b, u)}\right]=\left[\begin{array}{cccc}
S_{1}^{\prime} & S_{2}^{\prime} & -S_{1}^{\prime} & -S_{2}^{\prime} \\
& S_{3}^{\prime} & -S_{2}^{\prime} & -S_{3}^{\prime} \\
& & S_{1}^{\prime} & S_{2}^{\prime} \\
& & S_{3}^{\prime}
\end{array}\right]}  \tag{8}\\
& \left\{\begin{array}{l}
S_{1}{ }^{\prime}=\frac{E A \cos ^{2} \alpha}{l} \\
S_{2}{ }^{\prime}=\frac{E A \sin \alpha \cos \alpha}{l} \\
S_{3}{ }^{\prime}=\frac{E A \sin ^{2} \alpha}{l}
\end{array}\right. \tag{9}
\end{align*}
$$

where $E, l$, and $A$ are the elastic modulus (MPa), length (m), and cross-sectional area $\left(\mathrm{m}^{2}\right)$ of the rod elements, respectively. $A$ represents the angle of the rod element between the LCS and the GCS $\left({ }^{\circ}\right)$.

Due to the discontinuity between rock layers, the elastic modulus and Poisson's ratio in the normal and tangential directions of the layered rock mass are distinguished. Specific values can be obtained directly through on-site testing. The elastic resistance coefficient can be calculated by using the Gallerkin formula [46]. Based on the local deformation theory, the stiffness matrix of the surrounding rock elastic resistance spring element is constructed in the GCS, as shown in Equations (10) and (11).

$$
\begin{gather*}
{\left[k_{(r, u)}\right]=\left[\begin{array}{lll}
S_{1}^{\prime \prime} & S_{2}^{\prime \prime} & 0 \\
& S_{3}^{\prime \prime} & 0 \\
& & 0
\end{array}\right]}  \tag{10}\\
\left\{\begin{array}{l}
S_{1}^{\prime \prime}=k_{n} l b \cos ^{2} \theta+k_{\tau} l b \sin ^{2} \theta \\
S_{2}^{\prime \prime}=k_{\tau} l b \sin \theta \cos \theta-k_{n} l b \sin \theta \cos \theta \\
S_{3}^{\prime \prime}=k_{n} l b \sin ^{2} \theta+k_{\tau} l b \cos ^{2} \theta
\end{array}\right. \tag{11}
\end{gather*}
$$

where $k_{n}$ and $k_{\tau}$, respectively, represent the elastic resistance coefficients of the rock mass $\left(\mathrm{kN} / \mathrm{m}^{3}\right)$ in the normal and tangential directions. $l$ and $b$ represent the circumferential and longitudinal influence lengths ( m ) of the spring element, respectively. $\theta$ represents the angle of the spring element between the LCS and GCS $\left({ }^{\circ}\right)$, which can be adjusted according to the strata inclination of the revealed layered rock mass during excavation.

The vertical and horizontal elastic resistance coefficients of the foundation rock are related to the rock strength, foundation friction, and rock compressibility, etc. Values can be obtained through field testing or by consulting relevant specifications. Based on the local deformation theory, the stiffness matrix of the foundation elastic support unit in the GCS is shown in Equations (12) and (13).

$$
\begin{gather*}
{\left[k_{(d, u)}\right]=\left[\begin{array}{ccc}
S_{1}^{\prime \prime \prime} & 0 & 0 \\
& S_{2}^{\prime \prime \prime} & 0 \\
& & S_{3}^{\prime \prime \prime}
\end{array}\right]}  \tag{12}\\
\left\{\begin{array}{l}
S_{1}^{\prime \prime \prime}=k_{f} b d \\
S_{2}^{\prime \prime \prime}=k_{d} b d \\
S_{3}^{\prime \prime \prime}=\frac{1}{12} k_{d} b d^{3}
\end{array}\right. \tag{13}
\end{gather*}
$$

where $k_{f}$ and $k_{d}$ represent the vertical and horizontal elastic resistance coefficients of the foundation rock $\left(\mathrm{kN} / \mathrm{m}^{3}\right)$. $b$ represents the longitudinal influence length of the base elastic support (m). $d$ represents the width of the base elastic support (m).

### 3.2. Overall Stiffness Matrix of Support Structure

The equilibrium equations are formulated based on the equilibrium conditions at each node, as shown in Equation (14). It should be noted that the stiffness matrix [ $k_{r}$ ] of the surrounding rock elastic resistance where the spring elements are under tension should be treated as a zero-matrix.

$$
\left\{\begin{array}{l}
\left\{P_{1}\right\}=\left[\left[k_{s 11}\right]^{(1)}+\left[k_{r}\right]+\left[k_{b}\right]+\left[k_{d}\right]\right]\left\{\Delta_{1}\right\}+\left[k_{s 12}\right]^{(1)}\left\{\Delta_{2}\right\}  \tag{14}\\
\left\{P_{2}\right\}=\left[k_{s 21}\right]^{(1)}\left\{\Delta_{1}\right\}+\left[\left[k_{s 22}\right]^{(1)}+\left[k_{s 22}\right]^{(2)}+\left[k_{r}\right]\right]\left\{\Delta_{2}\right\}+\left[k_{s 23}\right]^{(2)}\left\{\Delta_{3}\right\} \\
\left\{P_{3}\right\}=\left[k_{s 32}\right]^{(2)}\left\{\Delta_{2}\right\}+\left[\left[k_{s 33}\right]^{(2)}+\left[k_{s 33}\right]^{(3)}+\left[k_{r}\right]\right]\left\{\Delta_{3}\right\}+\left[k_{s 34}\right]^{(3)}\left\{\Delta_{4}\right\} \\
\vdots \\
\vdots \\
\vdots \\
\vdots \\
\left\{P_{\mathrm{n}-1}\right\}=\left[k_{s \mathrm{n}-1 \mathrm{n}-2]^{(\mathrm{n}-2)}\left\{\Delta_{\mathrm{n}-2}\right\}+\left[\left[k_{s \mathrm{n}-1 \mathrm{n}-1}\right]^{(\mathrm{n}-2)}+\left[k_{s} \mathrm{n}-1 \mathrm{n}-1\right]^{(\mathrm{n}-1)}+\left[k_{r}\right]\right]\left\{\Delta_{\mathrm{n}-1}\right\}+\left[k_{s \mathrm{n}-1 \mathrm{n}}\right]^{(\mathrm{n}-1)}\left\{\Delta_{\mathrm{n}}\right\}}\right. \\
\left\{P_{\mathrm{n}}\right\}=\left[k_{s \mathrm{nn}-1}\right]^{(\mathrm{n}-1)}\left\{\Delta_{\mathrm{n}-1}\right\}+\left[\left[k_{s \mathrm{nn}}\right]^{(\mathrm{n}-1)}+\left[k_{r}\right]+\left[k_{b}\right]+\left[k_{d}\right]\right]\left\{\Delta_{\mathrm{n}}\right\}
\end{array}\right.
$$

Based on the element location and the node number, by substituting and superimposing the extended stiffness matrix of each element into the overall stiffness matrix, the overall stiffness matrix of the support structure can be obtained, as shown in Equation (15).

As tunnel excavation progresses, it is necessary to adjust the D-S load calculation model by adding and connecting the new primary support elements to the previous primary support structure. At the same time, the surrounding rock elastic resistance spring elements are applied to the new structure, and elastic base support elements are moved to the arch foot of the new structure. Until the primary support construction is completed and the elastic beam elements form a closed circular structure, the D-S load calculation model for the entire construction stage has been established, as shown in Figure 3.


Figure 3. The D-S load calculation model in different construction stages: (a) middle bench; (b) lower bench; and (c) complete structure.

### 3.3. Calculation of Equivalent Node Load

The overall stiffness matrix of the D-S load calculation model is a positive-definite, sparse, and banded matrix, which has obvious symmetry and singularity. Based on the unique deformation values at the key nodes, the real solution for the corresponding node forces can be obtained. After measuring the deformation values at the key nodes of typical sections and subtracting the overall displacement of the structure, the node displacement matrix of the support structure ( $\left[\Delta_{i}\right]_{n \times 1}$ ) can be formed. Substituting the overall stiffness matrix and node displacement matrix into Equation (3) and subtracting the weight of the support structure, the loads acting on the primary support structure at each key node are obtained. Loads at other positions can be calculated by linear interpolation. Then, the calculation is repeated after the tensile spring elements have been removed until there are no tensile spring elements. Finally, the reasonable equivalent nodal load matrix of the primary support structure can be obtained. The distribution form of the loads on the tunnel structure can be determined.

## 4. Calculation Examples and Results

The Muzhailing Super-long Highway Tunnel (abbreviated as Muzhailing Tunnel) is a key project of the Lanzhou-Haikou National Expressway (G75) in northwest China. The tunnel has two unidirectional bores and four lanes. The lengths of its left and right tunnels are $15,231 \mathrm{~m}$ and $15,173 \mathrm{~m}$, respectively. The maximum buried depth of the tunnel is about 629.1 m , making it a deep-buried tunnel. The tunnel has a clearance width of 10.25 m and a clearance height of 5.0 m . The tunnel is designed with a speed limit of $80 \mathrm{~km} / \mathrm{h}$. Three inclined shafts are arranged, among which, the inclined shaft No. 3 is 1265 m long, with an average longitudinal slope of $-12.7 \%$. The geographical location and longitudinal cross-section of the Muzhailing Tunnel and inclined shaft No. 3 are shown in Figure 4. The main stratum lithology comprises grey calc-siliceous sandy slate and black carbonaceous phyllite. Two kinds of rocks are distributed in different proportions and thicknesses, with significant layered characteristics in monoclinic structures. In this geological condition, asymmetric deformation disasters of the support structure occur frequently. Therefore, taking the typical deep-buried layered soft rock section of the Muzhailing Tunnel as the research object, the D-S load calculation model is adopted to explore the magnitude and distribution form of the load acting on the tunnel structure.


Figure 4. Geographical location and longitudinal cross-section.
Monitoring measurements were carried out on the typical deep-buried layered soft rock section of the inclined shaft No. 3 (K1+132, K1+140 and K1+163). The strata thicknesses of the $\mathrm{K} 1+132, \mathrm{~K} 1+140$, and $\mathrm{K} 1+163$ sections are $10-20 \mathrm{~cm}, 5-15 \mathrm{~cm}$, and $15-25 \mathrm{~cm}$, respectively. The strata inclinations of those sections are $60-70^{\circ}, 40-50^{\circ}$, and $20-30^{\circ}$, respectively. Seven key monitoring points were set up at those sections, including the vault, left and right haunch, left and right arch foot of the upper step, and left and right arch feet of the middle step. Vertical settlement and horizontal convergence values were measured at each point to form a node displacement matrix of the primary support structure. The deformation curve and distribution diagram of the monitoring sections are shown in Figure 5.

According to on-site tests and reviews of engineering geological exploration data [47-49], the basic parameters were calculated and obtained, as shown in Tables 1-5.

Table 1. Basic calculation parameters.

| Tunnel |  |  |  | Surrounding Rock |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Burial Depth (m) | Span (m) | Height (m) | Radius (m) | Weight <br> $\left(\mathbf{k N} / \mathbf{m}^{\mathbf{3}}\right)$ | Cohesion (kPa)Internal Friction <br> Angle ( ${ }^{\circ}$ ) |  |
| 455 | 12.5 | 11.6 | 6.15 | 25 | 200 | 25 |

Table 2. Parameters of elastic beam element.

| Dimension (m) | Length (m) | Moment of <br> Inertia $\left(\mathbf{m}^{4}\right)$ | Equivalent Elastic <br> Modulus (GPa) | Equivalent <br> Gravity $\left(\mathbf{k N} / \mathbf{m}^{3}\right)$ | Poisson's Ratio |
| :---: | :---: | :---: | :---: | :---: | :---: |
| $0.5 \times 0.25$ | 1.0 | $6.51 \times 10^{-4}$ | 29.5 | 23.7 | 0.2 |


(a)


(b)


Figure 5. Deformation curve and distribution diagram: (a) K1+132 Section; (b) K1+140 Section; and (c) K1+163 Section.

Table 3. Parameters of elastic rod element.

| Dimension (m) | Length (m) | Equivalent Elastic <br> Modulus (GPa) | Equivalent Gravity <br> $\left(\mathbf{k N} / \mathbf{m}^{\mathbf{3}}\right)$ | Poisson's Ratio |
| :---: | :---: | :---: | :---: | :---: |
| $0.04 \times 0.04$ | 1.0 | 206 | 78.5 | 0.3 |

Table 4. Parameters of elastic resistance spring element.

| Circumferential Influence Length (m) | Longitudinal Influence Length (m) | Normal Spring Stiffness Coefficient ( $\mathrm{kN} / \mathrm{m}^{3}$ ) | Tangential Spring Stiffness Coefficient ( $\mathrm{kN} / \mathrm{m}^{3}$ ) | Vertical Rock Layers |  | Parallel Rock Layers |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  |  | Elastic Modulus (GPa) | Poisson's Ratio | Elastic Modulus (GPa) | Poisson's Ratio |
| 1.0 | 0.5 | $2.34 \times 10^{5}$ | $1.7 \times 10^{5}$ | 12 | 17 | 0.15 | 0.18 |

Table 5. Parameters of base elastic support element.

| Longitudinal Influence <br> Length $(\mathbf{m})$ | Width of Support Structure <br> Base $(\mathbf{m})$ | Vertical Elastic Resistance <br> Coefficient $\left(\mathbf{k N} / \mathbf{m}^{3}\right)$ | Horizontal Elastic Resistance <br> Coefficient $\left(\mathbf{k N} / \mathbf{m}^{3}\right)$ |
| :---: | :---: | :---: | :---: |
| 0.5 | 0.28 | $2 \times 10^{5}$ | $1.5 \times 10^{5}$ |

The equivalent node load in the GCS calculated by the D-S load calculation method was converted into a uniformly distributed load of the element in the LCS. The magnitude and distribution of the load and elastic resistance acting on the primary support structure were obtained. The calculation results are shown in Figures 6-8.


Figure 6. Calculation results of the K1+132 Section: (a) load; (b) surrounding rock pressure; and (c) elastic resistance.

(a)

 - Construction completion of lower bench - Construction completion of primary support
(b)


- Construction completion of upper bench - Construction completion of middle bench
-Construction completion of lower bench
- Construction completion of primary support
(c)

Figure 7. Calculation results of the K1+140 Section: (a) load; (b) surrounding rock pressure; and (c) elastic resistance.

According to the calculation results, the load distribution was significantly related to the deformation trend of the support structure and the strata inclination of the rock layer. Overall, the distribution form showed that the left side was greater than the right side, and the upper bench was greater than the middle bench. Taking the K $1+140$ section as an example, the maximum load ratio on the left and right sides at the same horizontal reached 16.62:1. The maximum load ratio of the upper and middle benches reached 2.73:1. It could be seen that the load differences in various parts of the deep-buried layered soft rock tunnel were very significant. As the strata inclination of the rock layer gradually decreased, the maximum load of the $\mathrm{K} 1+163$ section was reduced by $63.52 \%$ compared to the K1+140 section. The non-uniform load decreased and gradually tended to a symmetrical
distribution. In addition, the load was greatly affected by the construction process. As the excavation of various parts of the tunnel progressed, the load and elastic resistance continued to increase. Similarly, taking the K1 +140 section as an example, the load variation ratio in the left arch foot of the upper step during the upper bench, middle bench, lower bench, and inverted arch stages was 5.52:5.01:1.28:1. It can be seen that the increase in load varied greatly in different construction stages. Until the construction of the support structure was completed, the rate of the load growth decreased and eventually reached a stable state. At this time, the load distribution was extremely uneven, and the phenomenon of tunnel bias was very significant.


Figure 8. Calculation results of the K1+163 Section: (a) load; (b) surrounding rock pressure; and (c) elastic resistance.

Considering an analysis from the perspective of structural stress, the maximum load acting on the primary support structure ranged from 0.473 MPa to 1.298 MPa . For thinwalled large-span arch structures such as primary supports, the load at this range exceeded the ultimate buckling load value of the structure [50]. In addition, the non-uniformity and asymmetry of the load could easily cause unilateral local large deformation of the tunnel structure. A serious asymmetric large deformation disaster occurred in the inclined shaft No. 3 of the Muzhailing Tunnel, as shown in Figure 9. Therefore, the distribution of and variation in the calculation load can reasonably explain the deformation disasters on one side of the primary support in actual construction, such as the cracking and spalling of the shotcrete, the bending and twisting of steel frames, and instability of and damage to the support structure.


Figure 9. Asymmetric large deformation disasters of the inclined shaft No. 3.

## 5. Discussion

### 5.1. Analysis of Sample Data for Layered Rock Tunnels

A total of 182 sets of sample data on loads and deformations for 56 typical deepburied layered soft rock tunnels during their construction were collected through field measurements and literature research, mainly including the Muzhailing railway tunnel, the Erlang Mountain tunnel, the Zhegu Mountain tunnel, the Chhibro-Khodri tunnel, the Udhampur railway tunnel, and the Chenani-Nashri tunnel [51-56]. Based on the load distribution range under different deformation values, the reliability of the D-S load
calculation method was verified. After excluding individual special sample data, the sample data were fitted using power and exponential functions, as shown in Figure 10.


Figure 10. Cross scatter envelope diagram of the sample data.
Affected by objective factors such as tunnel support measures, construction methods, and monitoring methods, the sample data are relatively scattered. The upper and lower envelope ranges are relatively large. However, the sample data distributed above and below the overall regression formula account for $53.77 \%$ and $46.23 \%$ of the total sample data, respectively. The quantities of the two are basically the same. Moreover, the sample data distributed around the overall regression formula are relatively concentrated, indicating that the overall regression formula can represent the average level of the load under different deformation values. Then, an $80 \%$ enveloping degree of the sample data [57] is selected to evaluate the reliability of the D-S load calculation method. It can be found that the load values calculated by the D-S load calculation method are distributed in this region. Since this method considers the influence of the geometric shape of the structure in different construction stages on the load, the trend of the load change with deformation is slightly different from that of the overall regression formula. Nonetheless, the calculation results are relatively close to the overall regression formula. On the other hand, the discreteness of the sample data is mainly due to the complex engineering characteristics of deep-buried layered soft rock tunnels. The structural shape, surrounding rock parameters, and material parameters of different tunnel projects are not the same. Thus, some typical tunnel projects were selected to calculate the load by using the D-S load calculation method. From the calculation results, it can be seen that the relative error from the field-measured and calculation values under the same geological conditions is less than $10 \%$. The fitted curves are distributed between the upper and lower envelope lines. This indicates that the loads calculated by the D-S load calculation method are in a reasonable range and that the results are reliable.

### 5.2. Comparison and Analysis of Different Load Calculation Methods

The M. M. Promojiyfakonov theory, Terzaghi theory, Specifications methods, Q-system methods, Fenner formula, Kommerall method, and Бугаева method are selected to calculate the rock pressure and elastic resistance of a typical section of the inclined shaft No. 3 in the Muzhailing Tunnel, as shown in Figure 11. The characteristics and advantages of the D-S load calculation method are analyzed by comparing the calculation results of these various methods.

The calculated results of the D-S load calculation method differ significantly from those of the Promojiyfakonov theory, Terzaghi theory, Specifications methods, and Fenner formula. The main reasons for this difference are as follows: (1) The Promojiyfakonov theory and Terzaghi theory only reflect the state of loose rock mass through the cohesive force and internal friction angle. Specifications methods reflect the characteristics of the surrounding rock by defining the level, while the Fenner formula reflects the state of the deep rock mass through the initial geostress value and the plastic failure range after tunnel excavation. The influencing factors considered by the above methods are relatively simple and struggle to reflect the complexity of the engineering characteristics of deep-buried layered soft
rock tunnels. (2) The selection of calculation parameters for the Promojiyfakonov theory, Terzaghi theory, and Specifications methods is based on the overall geological conditions of the tunnel site. Under complex and variable surrounding rock conditions, the range of calculation parameters is inaccurate, and it is difficult to characterize the physical and mechanical properties of the rock mass at specific cross-sections. In contrast, the Q-system method comprehensively and in detail describes the various factors affecting the surrounding rock pressure by using a large number of parameters, such as the quality index of the rock mass, volumetric joint count of the rock mass, and joint surface roughness, etc. It follows that the calculated values from the Q-system method are close to those of the D-S load calculation method. Moreover, the calculated results of the D-S load calculation method are significantly smaller than those obtained by using the Kommerall method and Бугаева method. This is because traditional methods are all based on the assumption of elastic resistance distribution, which has a weak synergy with the actual stress situation of the structure. Additionally, the cross-section of the Muzhailing Tunnel adopts a horseshoe shape that is different from circular or elliptical shapes. They ignore the influence of the elastic modulus and the cross-sectional size of the tunnel lining structure on the elastic resistance. For this reason, the applicability of these methods is poor under deep-buried layered geological conditions.


Figure 11. Calculation results of various methods.
Compared with the above-mentioned load calculation methods, the D-S load calculation method has advantages such as easy parameter acquisition, close integration with the engineering characteristics of deep-buried layered soft rock tunnels, and strong targeting of the calculation results. Various parameters used in the calculation process can be directly obtained based on material properties, on-site measurements, or mechanical calculations. Among them, tunnel deformation, as one of the most common and essential monitoring items during construction, is a comprehensive indicator that reflects the objective factors of tunnel engineering, external factors of construction, and subjective factors of surrounding rock. The load acting on the structure is calculated according to the structural deformation, and many factors such as the crustal stress state, mechanical property of the rock mass, and geological characteristics of the surrounding rock can be considered. Moreover, this method considers the influence of construction factors on loads, such as cross-sectional shape, excavation procedures, and disturbance. By adjusting the overall stiffness matrix of the tunnel support in a timely manner, the dynamic monitoring of structural loads during tunnel construction can be realized, so as to reinforce parts with excessive loads and reduce the further development of deformation disasters in the early stage of incubation. Furthermore, this method can accurately calculate the load distribution at each position of a specific section of the tunnel. Accordingly, the support parameters of the tunnel can be
designed asymmetrically, avoiding deformation disasters caused by unilateral damage to the structure. However, due to the consideration of structural geometric dimensions and construction methods in the D-S load calculation method, it is necessary to reconstruct the geometric form of the model and adjust the position and quantity of various units according to specific processes, in order to promote its application in other tunnel projects. Therefore, the D-S load calculation method is more suitable for long and large tunnel projects. After the model is established, only deformation data are needed to obtain the load values at different cross-sections.

### 5.3. Application and Prospects

On the basis of determining the reliability of the D-S load calculation method, combined with the characteristics of this method, outlook and analysis are conducted on the application prospects of this method in similar geological environments, as shown in Figure 12, in order to provide directions for future research.


Figure 12. Application and prospects of the D-S load calculation method.
Deformation data are the key and prerequisite for the D-S load calculation method, which require researchers to monitor and measure on construction sites. This can obtain reliable deformation data to ensure the accuracy of the load calculation results. In addition, considering that the calculation process of the D-S load calculation method is relatively fixed, the corresponding stiffness matrix will not change after determining the structural shape, material parameters, surrounding rock parameters, and construction procedures, etc. Thus, visual programs can be written and created to build a tunnel simulation platform. After obtaining deformation monitoring data, the simulation platform can be used to achieve the real-time monitoring of loads. Furthermore, numerical simulation functions can be loaded on this platform to predict the stress on the support structure. Based on the mechanical behavior of the structure, the design parameters of the lining structure can be adjusted and finally provide feedback for the actual construction of the tunnel. This can enable the timely reinforcement of the tunnel structure where structural damage may occur, ensuring the efficiency, economy, and safety of tunnel construction.

## 6. Conclusions

In this paper, a load calculation method was proposed based on structural deformation in deep-buried layered soft rock tunnels. Firstly, by building the D-S load calculation model and considering the impacts of tunnel excavation methods and construction processes, the magnitude and distribution of loads acting on the tunnel structure were calculated. Then, the reliability of the D-S load calculation method was verified by investigating the relationship between deformation and load in typical deep-buried layered soft rock tunnels.

Additionally, this method was compared with various classic load calculation methods, and its advantages in deep-buried layered soft rock tunnels were analyzed. Finally, the application prospects of this method in similar geological environments were discussed and analyzed. The main conclusions are summarized as follows.
(1) Based on the mechanical properties of the tunnel support structure material, the stiffness matrix of each element was formed. Meanwhile, considering the geometric shape and corresponding constraints of the section under different construction processes, the D-S load calculation model for the primary support was established. The load acting on the tunnel structure and its distribution form can be obtained according to the structural deformation by using this model.
(2) Taking the inclined shaft No. 3 of the Muzhailing Tunnel as an example, on-site monitoring and measurements of specific sections were carried out. Based on the measured deformation values and relevant parameters, the loads acting on the support structures were calculated by using the D-S load calculation method. According to the calculation results, deep-buried layered soft rock tunnel sections bore a certain degree of bias pressure. The load distribution gradually tended toward symmetric as the strata inclination of the rock layer decreased. The load value gradually increased as the tunnel excavation progressed. From a mechanical perspective, a reasonable explanation was given for the phenomenon of localized damage on one side of the tunnel, providing a theoretical basis for research on the mechanisms of asymmetric large deformation and targeted prevention and control measures in deep-buried layered soft rock tunnels.
(3) The reliability of the D-S load calculation method was verified by fitting and analyzing the sample data of the deformation and load of typical deep-buried layered soft rock tunnels. Compared with existing load calculation methods, the D-S load calculation method can obtain the load distribution at different positions of a specific tunnel section, which is more suitable for calculating the load of deep-buried layered soft rock tunnels.
In summary, considering the excellent reliability and accuracy of the D-S load calculation method, the construction of a tunnel simulation platform in future research is considered. Through the real-time monitoring of deformation data, the load acting on the support structure can be accurately calculated and fed back to the tunnel design. The support scheme and parameters can be adjusted in a timely manner to achieve the efficient, economical, and safe construction of deep-buried layered soft rock tunnels.

Author Contributions: Conceptualization, L.Z., L.C., J.C., Y.L. and H.G.; methodology, L.Z., L.C. and J.C.; validation, L.Z., L.C., J.C., Y.L. and H.G.; formal analysis, L.C.; investigation, Y.Z.; data curation, L.Z., Y.Z. and P.W.; writing-original draft, L.Z. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by Project of National Natural Science Foundation of China, grant number 41831286.
Data Availability Statement: The data that support the findings of this study are available within the manuscript.

Conflicts of Interest: The authors declare no conflicts of interest.

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