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Intelligent safety assessment method for demolition construction of closure segment of long-span continuous rigid frame bridges

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Abstract

In order to clarify the risk of demolition construction of large-span continuous rigid structure bridge and put forward an intelligent safety assessment method to ensure the safety of demolition construction of the closure segment. Taking a concrete continuous rigid bridge as an example, this paper uses the combination of finite element analysis, theoretical calculation and actual measurement verification to study the influencing parameters and construction safety assessment methods of the long-span continuous rigid bridge in the demolition construction stage of the closure segment. The results show that the parameters that have a great influence on the stress state of box girder and pier during the demolition stage of the closure segment are mainly the self-weight of the structure, tendon prestress state and construction temperature difference. Through the influence envelope analysis of each parameter, it is clear that the ultimate failure mode caused by the most unfavorable parameter combination in the demolition stage of the closure segment is the crushing of the bottom plate of the box girder in the middle span, and the cracking of the piers on the side span at the top and the variable section. In order to further accurately evaluate the construction safety in the demolition stage of the closure segment, based on the long-term down-warping state inversion analysis of the box girder, the identification method of cross-section damage and prestress loss of the box girder and the calculation results of engineering examples are given. Finally, a safety assessment method of the most unfavorable section based on the principle of influence matrix is proposed. Through the analysis of an example, the safety of the closure segment demolition construction is clarified, and the correctness of the analysis is verified by intelligent monitoring means.

Keywords: Bridge engineering, Continuous rigid frame bridge, Intelligent security assessment, Demolition of closure segment, Influence parameters

1 Introduction

Early prestressed concrete bridges were affected by multiple factors such as theoretical defects, construction technology, technical level, material performance, etc. After completion, problems such as cracking, prestress reduction and down-warping in mid-section generally occurred, affecting the durability and aesthetics of the structure, limiting the normal use of the bridge, and even causing the structural bearing capacity not to meet the requirements (Chen et al. 2022; Liu et al. 2022a, b). However, due to the limitations of bridge detection technology, it is difficult to identify the damage of bridge structure and cannot accurately quantify the analysis, which makes the prediction of bridge demolition construction status full of uncertainty (Zhao et al. 2007; Li and Wang 2019). In particular, the demolition of the closure segment of the continuous rigid frame bridge will result in system transformation, and the large deviation between the construction state simulation analysis and the measured results will lead to the difficulty in ensuring the safety of the demolition construction.

The general bridge safety assessment is mainly based on the finite element numerical simulation or actual bridge test, and the whole process analysis is carried out by using the method of safety index comparison (Liang et al. 2019). Liu et al. proposed a safety assessment method based on the internal force envelope theory with reference to the bridge design verification method, which can reflect the overall state of the bridge through finite measurement points (Liu et al. 2009). Zhang et al. based on the Bei-jiang Bridge of Fo-kai Expressway, used MIDAS program to simulate and analyze the stress and deformation of the bridge in the process of jacking demolition, and put forward relevant construction improvement measures according to the construction safety requirements (Zhang et al. 2013). To evaluate the bridge safety conditions comprehensively and scientifically, Li et al. artificial intelligence methods and data fusion techniques based on information entropy, fuzzy analytical hierarchy, and the Dempster-Shafer theory are utilized to establish the data processing unit (Li and Wang 2019). Sun et al. proposed to use the risk communication method to qualitatively identify the construction risk sources, use the numerical simulation analysis method to evaluate the safety of the construction, and invert and correct the model according to the measured data to predict the subsequent construction impact (Sun and Meng 2022).

For the old bridge, the real calculation parameters are often different from the theoretical values. The parameter error can be divided into two categories according to the cause, one is the structural damage in the long-term operation stage, and the other is the lack of accuracy in the construction stage. Shi et al. analyzed the influence of structural damage on natural frequency and vibration mode by analytical method, and pointed out that the element damage has little influence on the array vector, while the element damage at the support or at the end of the structure with torsional constraint has significant influence on natural frequency (Shi and Zhao 2007). Sun et al. took a concrete-filled steel tube arch bridge as an example, and compared the structural effects of component damage under different working conditions through finite element analysis. The results show that component damage will affect the overall stiffness and stability of the structure, and will cause significant redistribution of internal forces in the structure (Sun et al. 2018). Yan R Z et al. pointed out that the construction accuracy will greatly affect the stress of the structure by conducting sensitivity analysis on the error of construction parameters. In order to accurately understand the true stress state of the structure, it is necessary to detect and identify the values of relevant parameters (Yan et al. 2015). At present, bridge damage recognition research mainly extracts structural features (such as modal parameters, modal curvature, etc.) from time domain signals (such as acceleration time history, etc.), and then mines damage information from them through pattern recognition and machine learning methods (Meng et al. 2019; Shan et al. 2020). According to the vehicle bridge coupling model, Tan, Liu et al. used the extracted vibration signal and bridge modal shape to detect the local damage of the bridge, and the effect was good (Tan et al. 2020; Liu et al. 2020). Obrien et al. used deflection, beam end angle and acceleration response as local damage indicators for concrete bridge structures in operation, and found that there was an obvious nonlinear correlation between damage indicators and temperature excitation (Obrien et al. 2020). Khandel et al. proposed a damage identification method for prestressed concrete beams based on fiber bragg grating sensors, which detected structural damage without detailed load information by establishing the relationship between the strain response distribution of different measurement points on the beam, and verified the correctness of the proposed method through a large prestressed concrete bridge structural test (Khandel et al. 2021).

In order to ensure the safety of the demolition construction of the closure segment of the long-span continuous rigid bridge, this paper analyzes the error range of the calculation parameters of the continuous rigid bridge and the influence degree of the value range of different parameters by finite element analysis, so as to determine the influence degree of each parameter. At the same time, the potential risk area of the structure in the demolition stage of the closure segment is identified by the most unfavorable parameter combination envelope, and the safety assessment is carried out by the influence matrix method.

2 Structural security assessment for closure segment demolition construction of long-span continuous rigid frame bridge

2.1 Analysis of theoretical calculation parameter error values

Continuous rigid bridges often have problems such as insufficient construction accuracy and long-term operation damage of the structure, which will lead to differences between the actual parameters of the structure and the design values (Wang 2004; Hou and Wang 2021; Zhang et al. 2023). Figure 1 shows the structural performance deterioration curve of the bridge. As shown in the Fig. 1, the bridge has undergone operational damage and maintenance reinforcement since its construction, and its mechanical performance differs significantly from the original structure.

List the relevant calculation parameters that may affect the stress state of the structure, and take the structural self weight, material elastic modulus, pre-stress, construction temperature difference, structural section characteristics, etc. as the main analysis objects. The above parameters can be divided into construction error and performance

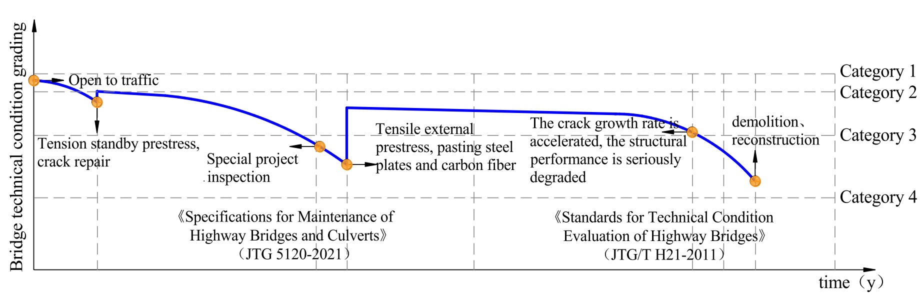

Fig. 1 Structure performance deterioration curve of bridge

deterioration according to the source of error, among which the self-weight of the main beam, the degree of prestressed anchorage, structural strength, construction temperature difference are classified as construction errors, which are mainly caused by insufficient construction accuracy. The upper limit of effective prestress, material elastic modulus and other parameters can also be considered as the result of construction error, while the lower limit is mainly attributed to the decline of parameter indicators caused by structural damage and material performance deterioration during operation.

The self-weight error of the structure is considered to be the deviation of concrete pouring square caused by the formwork positioning, deformation and elevation control errors, and the self-weight error range of the main beam is -5 ~ 10%. The cross-section moment of inertia error is mainly considered to be the effect of structural cracking in the operation stage, and the maximum reduction is -15%. The upper limit of effective prestress is considered to be construction over-tension (±5% according to the specification), while the lower limit is considered to be long-term prestress loss (-25% reduction of tension force). The anchoring degree of the prestressed tendon after cutting is affected by the quality of duct grouting. The anchoring degree range is 0 ~ 100% according to the two conditions of full grouting and no grouting. The value of material properties is based on the processing error of ± 10%, and the deterioration of material properties is considered to be -10% (generally the concrete strength will not decrease with time). However, the temperature during the removal of the closure segment must be different from that during the pouring of the closure segment. The temperature difference is considered as ±20℃.

The values of the above parameters are mainly based on the relevant construction, design, and testing specifications, and at the same time consult the relevant old bridge inspection and research data, and the specific parameters and parameter values are summarized as follows (Table 1).

Among them, the structural strength error mainly affects the bearing capacity of the structure, while the error of other parameters mainly affects the stress state. In the following paper, the structural stress in the demolition stage of the closure segment of the continuous rigid bridge is used as an index to analyze the degree of influence of each

Table 1 Parameter value range

Parameter	Upper limit	Lower limit	Error range	Reason	Remarks
Self-weight	95%	110%	-5% ~ 10%	Exceeds and owes	construction error
Moment of inertia	85%	100%	-15% ~ 0%	cracking damage	performance degradation
Effective prestress	75%	105%	-25% ~ 5%	prestress loss	performance degradation
Anchoring degree-prestress	0%	100%	0% ~ 100%	Grouting quality	construction error
Modulus of Elasticity-C	80%	110%	-20% ~ 10%	material properties	performance degradation
Modulus of Elasticity-Ps	80%	110%	-20% ~ 10%	material properties	performance degradation
structural strength	90%	110%	-10% ~ 10%	material properties	construction error
Temperature difference	-20℃,	20℃	-20℃ ~ 20℃	environmental impact	construction error

parameter, and then the parameter combination envelope results are used to evaluate the structural safety of the demolition stage of the bridge closure segment.

2.2 Influence analysis of parameters on demolition construction of middle-span closure segment

2.2.1 Analysis method for the influence of parameter errors

The relevant analysis is mainly based on the finite element software, and it is first necessary to establish a complete construction stage model including construction, operation and demolition. According to the parameter classification and the timing of error formation, the parameter error during the construction period is analyzed by modifying the parameter value in the construction stage, which goes through three stages: new construction, operation and demolition. The parameter error during the operation period was modified during the operation period, and the superposition result of the accumulated stress in the new construction stage and the stress change in the operation and demolition stages is used for analysis.

Taking the self-weight of main girder as an example, the material properties of concrete are modified at the beginning of construction stage and the stress results are analyzed directly at the demolition stage of closure segment. Taking the elastic modulus of concrete as an example, the design parameter model 1 is used in the construction stage and the modified model 2 is used in the operation stage, and the stress result of the demolition stage can be calculated by (cumulative stress in the construction stage of model 1) plus (the stress change amount of model 2 in the operation and demolition stages).

2.2.2 Parameter error analysis of theoretical calculation

To analyze the influence degree of parameter errors on the stress level of continuous rigid frame bridges and their influencing patterns, and to clarify the possible ultimate failure modes under the most unfavorable parameter combinations, a prestressed concrete continuous rigid frame bridge with a main span of 106m is taken as an example. The main beam is a single-box single-chamber concrete box girder, with the beam height of 5.5m at the pier top and 3.1m at the mid-span. The beam bottom line is a quadratic parabola. The bridge adopts a three-way prestressing system, longitudinal and transverse steel strands, vertical rolled threaded crude steel bars. Among them, the construction scheme of the new formal installation stage is the symmetrical balanced hanging method of hanging basket section by section, and the segment cutting method of reverse pouring sequence is adopted for demolition construction. In order to analyze the influence degree of construction error in the demolition stage of closure segment and facilitate the structural safety assessment, the finite element model is established by MIDAS CIVIL, as shown in Fig. 2. The whole bridge adopts beam elements, and the boundary conditions are consolidated by pile bottom.

The calculation parameters of the theoretical model are all based on the design values, and the prestress effect is considered according to the failure after cutting. The error of self-weight and prestressed anchorage degree on demolition construction of middle-span closure segment is used as an example.

Concrete placing often has the problem of over-square, which can be caused by many reasons, such as size and position of formwork, content of reinforcement, deviation of

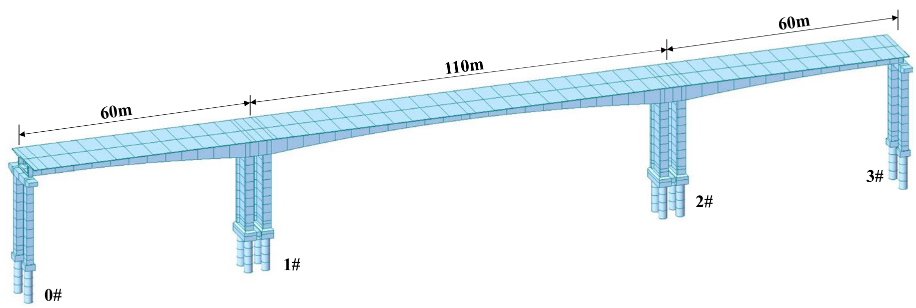

Fig. 2 Finite element model

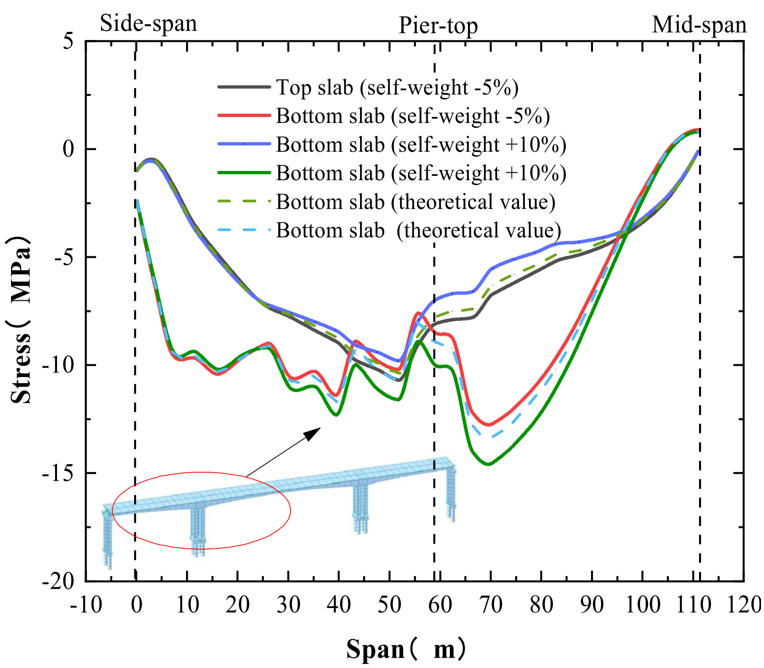

Fig. 3 Envelope diagram of the girder stress affected by the self-weight

elevation, etc. The requirements for the size of the structure in the specification are relatively strict, but the actual situation is generally difficult to accurately control, resulting in differences in the true self-weight of the structure. Figures 3 and 4 show the stress envelopes of main girders and piers in the demolition stage of the mid-span closure segment. It can be seen that the self-weight of the main girder increases, the compressive stress of the top slab of the main girder decreases while the compressive stress of the bottom slab increases, and the increment of the compressive stress of the pier top is greater than that of the bottom, and the mid-span side is greater than that of the side-span. When the self-weight of the girder decreases, the stress change law is reversed.

When the post-tensioned prestressed concrete bridge is dismantled, the prestressed tendons change from the end anchor to the bonded anchor after cutting, which can be approximately considered according to the force characteristics of the prestressed tendons. That is, within the prestress transfer length range, the prestressed tendon changes linearly from the zero-stress state at the end to the effective prestressed state

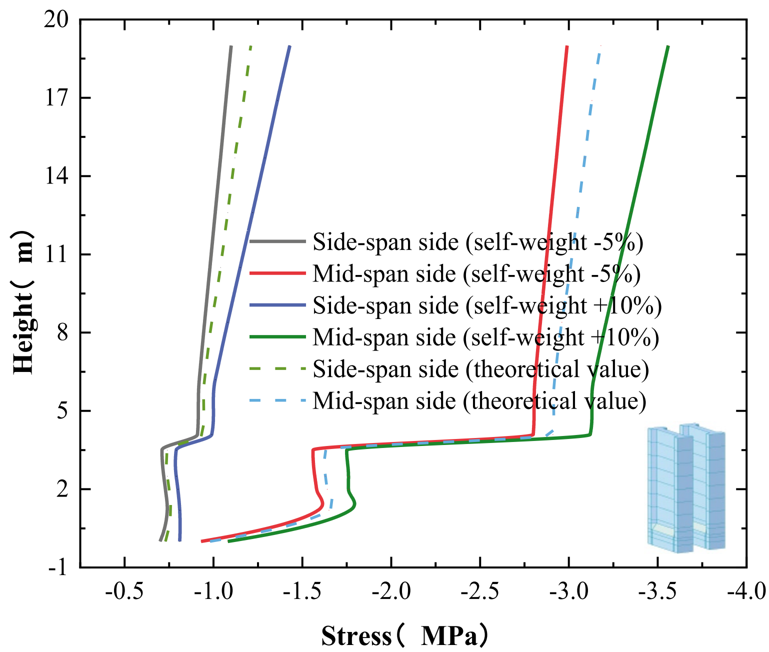

Fig. 4 Envelope diagram of the pier stress affected by the self-weight

through the bonding force, and continues to provide a positive bending moment effect on the main beam. However, the prestressed anchoring effect is affected by various factors such as prestressed structural damage, material performance degradation, and prestressed pore grouting quality. Therefore, the influence of the anchorage degree on the structure after the tendons is cut is analyzed below, and the two situations of complete anchorage and complete failure after prestress cutting are taken as envelope analysis.

It can be seen from Figs. 5 and 6 that the degree of prestressed anchorage has a significant effect on the stress of the bottom slab, and the compressive stress of the base slab is generated in the area adjacent to the cantilever section. For piers, prestressed anchoring increases the top compressive stress but reduces the reserve of compressive stress in the pier bottom area.

Referring to the analysis method of self-weight and prestress anchorage degree, the parameters such as loss of pre-stress, section damage and elastic modulus of material are analyzed (the maximum error of each parameter is compared), and the results of the influence of each parameter on the structural stress were obtained as follows (Figs. 7 and 8).

It can be found that the concrete mold does not affect the stress state of the structure, and other parameters have more or less influence on the structural stress to a certain extent. Among them, the parameters that have a great influence on the stress of the girder mainly include the self-weight, effective prestress, and the degree of prestressed anchorage, especially the degree of prestressed anchorage has a decisive influence on the stress of the bottom slab. The parameters that have a great influence on the pier are mainly effective prestress, prestress anchorage degree and construction temperature difference.

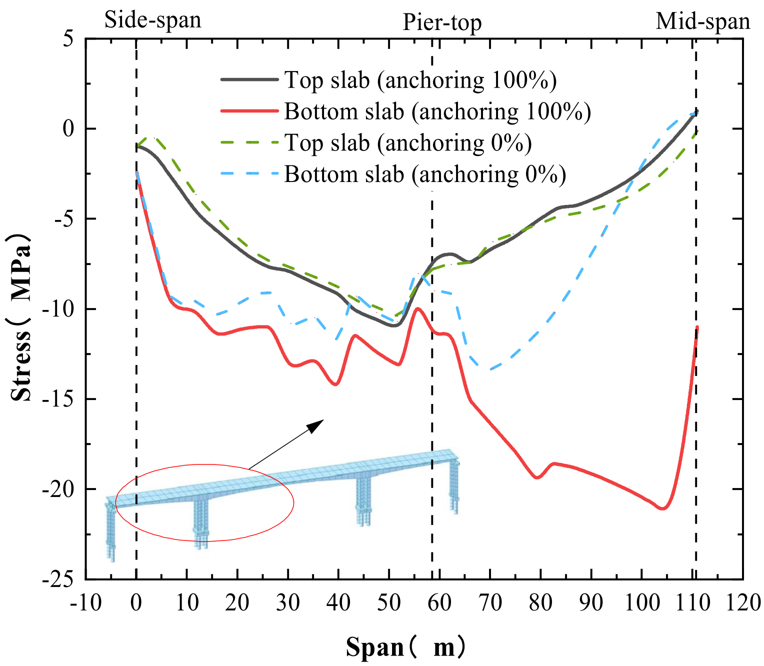

Fig. 5 Envelope diagram of the girder stress affected by the anchorage degree

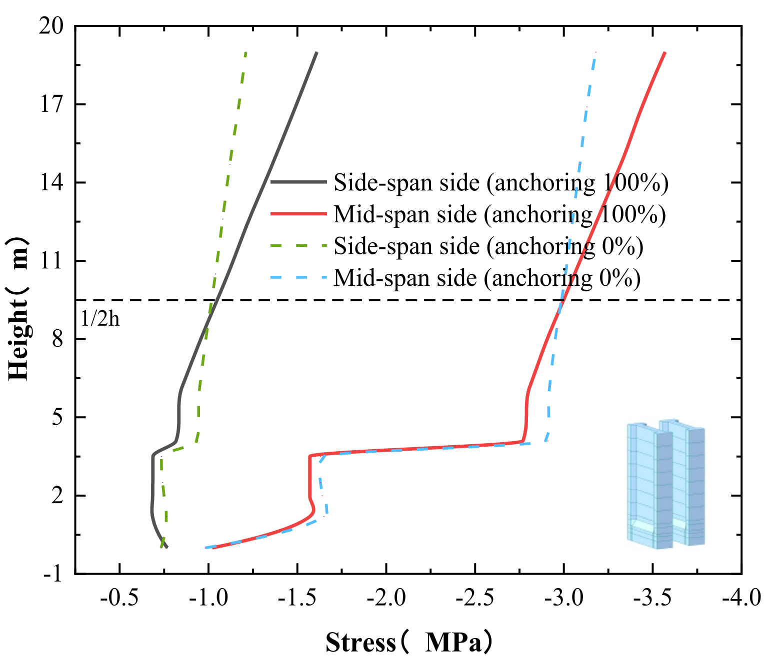

Fig. 6 Envelope diagram of the pier stress affected by the anchorage degree

By comparing the stress influence degree of each parameter, it can be found that some parameters, such as the degree of pre-stress anchorage, effective pre-stress, construction temperature difference, etc., have different influence laws on different positions of the structure. Therefore, the most unfavorable parameter combination effects at different positions should be considered separately in the analysis.

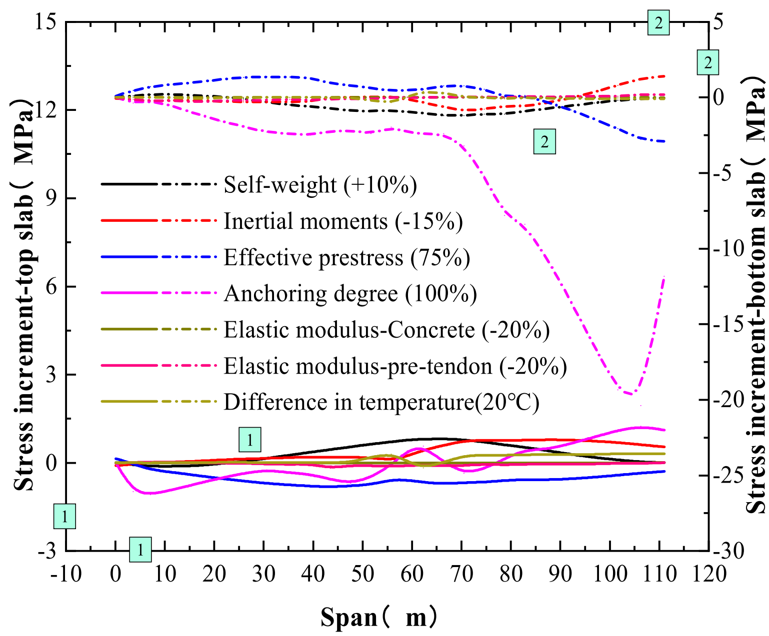

Fig. 7 Stress variation diagram of girder under influence of various parameters

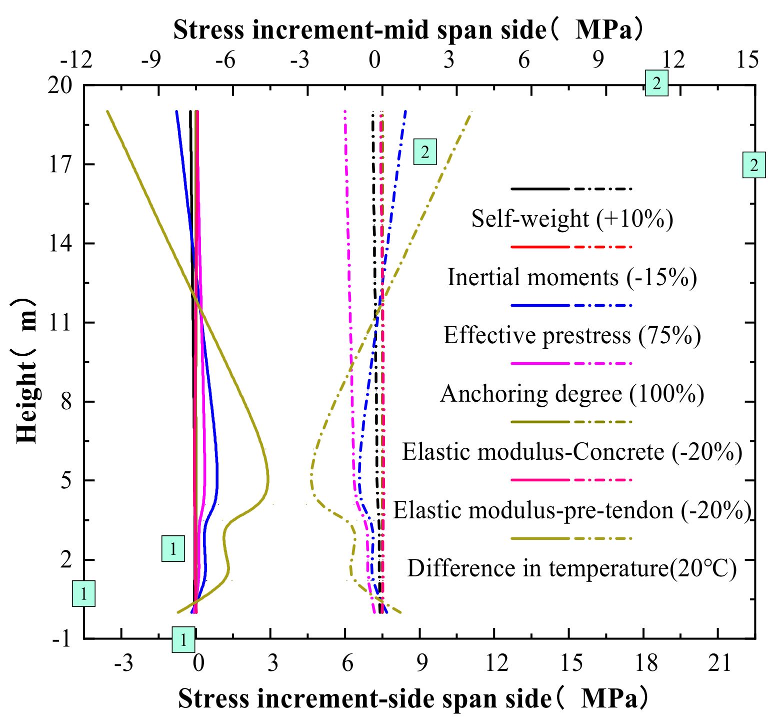

Fig. 8 Stress variation diagram of pier under influence of various parameters

2.3 Limit failure mode on demolition construction of middle-span closure segment

According to the influence analysis results of various parameters in the demolition stage, the stress influence law of the girder and pier is obtained as shown in Tables 2 and 3 ("increase" means that the compressive stress decreases or the tensile stress increases).

Table 2 Stress influence law of girder on demolition construction of middle-span closure segment

Structural position of girder	Top slab		Bottom slab	
Influence law	**Increase**	**Decrease**	**Increase**	**Decrease**
Self-weight	+10%	-5%	-5%	+10%
Moment of inertia	-15%	/	/	-15%
Effective prestress	/	75%	75% (side span)	75% (mid span)
Anchoring degree-prestress	100% (side span)	100% (mid span)	0%	100%
Modulus of Elasticity-C	/	/	/	/
Modulus of Elasticity-Ps	+10%	-20%	+10%	-20%
Temperature difference	+20℃	-20℃	-20℃	+20℃

Table 3 Stress influence law of pier on demolition construction of middle-span closure segment

Structural position of pier	Sid-span side		Mid-span side	
Influence law	**Increase**	**Decrease**	**Increase**	**Decrease**
Self-weight	-5%	+10%	-5%	+10%
Moment of inertia	/	/	/	/
Effective prestress	105% (2/3 h ~ h)	75% (2/3 h ~ h)	75% (2/3 h ~ h)	/
Anchoring degree-prestress	0% (1/2 h ~ h)	100% (1/2 h ~ h)	0% (1/2 h ~ h)	100% (1/2 h ~ h)
Modulus of Elasticity-C	/	/	/	/
Modulus of Elasticity-Ps	-20% (3/5 h ~ h)	+10% (3/5 h ~ h)	/	-20% (3/5 h ~ h)
Temperature difference	-20℃ (3/5 h ~ h)	+20℃ (3/5 h ~ h)	+20℃ (3/5 h ~ h)	-20℃ (3/5 h ~ h)

According to the influence law of each parameter in Tables 2 and 3, the parameters with the same influence law for each section position of girder and pier are enveloped to analyze the most disadvantageous section position on demolition construction of middle-span closure segment, so as to facilitate timely safety assessment during removal.

Taking the top slab of girder as an example, the stress envelope combination is as follows: the upper limit combination is: (Self-weight +10%) + (Moment of inertia -15%) + (Anchoring degree-prestress 100% in mid-span) / (Anchoring degree-prestress 0% in side-span) + (Modulus of Elasticity-prestress tendons +10%) + (Temperature difference +20℃); The lower limit combination: (Self-weight -5%) + (Effective prestress 75%) + (Anchoring degree-prestress 100% in side-span) / (Anchoring degree-prestress 0% in mid-span) + (Modulus of Elasticity-prestress tendons -20%) + (Temperature difference -20℃).

Figures 9 and 10 show the structural stress envelope under the most unfavorable combination of various parameters. For the demolition stage of the mid-span closure segment, under the influence of the most unfavorable parameter combination, there will be large compressive stress (prestressed anchorage effect) in the bottom plate near the mid-span cantilever section, and this area may be damaged due to the compressive stress exceeding the limit. At the same time, cracks may also occur at 1/5 height of the side span (variable section) and near the pier top due to excessive tensile stress. That is, the parts where ultimate damage may occur on demolition construction of middle-span closure segment are the bottom slab near the cantilever end of the beam, the variable section and top of the pier at the side span.

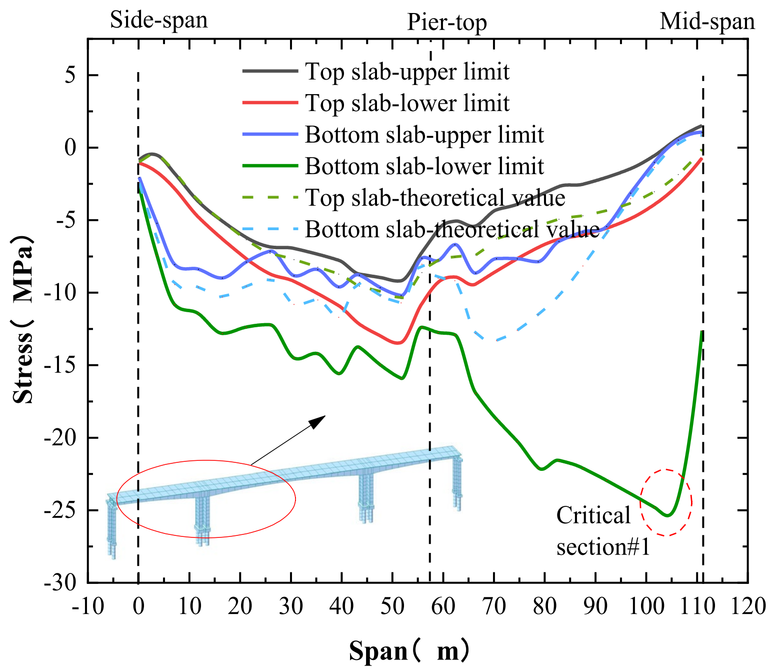

Fig. 9 Envelope diagram of the most unfavorable parameter combination of girder

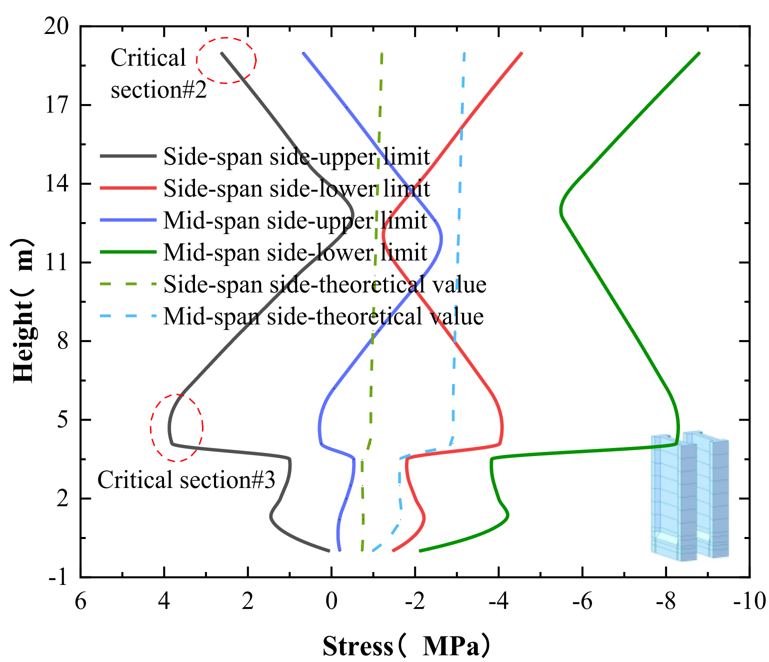

Fig. 10 Envelope diagram of the most unfavorable parameter combination of pier

3 Structural safety assessment method based on structural damage identification and impact matrix

According to the analysis of the influence of parameter errors, the typical ultimate failure mode of continuous rigid frame bridges under the influence of the most unfavorable parameter combination is the collapse of the mid span bottom plate of the main beam, and tensile cracking at the top and bottom of the bridge pier at the side span side pier and at the variable cross-section of the pier bottom. In order to further accurately evaluate the construction safety of the closure section demolition and determine the true stress situation of the actual bridge, it is necessary to conduct more accurate testing or identification of the values of relevant parameters.

3.1 Identification of structural damage parameters

The core of bridge demolition is to ensure that the construction state is safe and controllable, so that the demolition construction can proceed smoothly. Therefore, how to accurately evaluate the real parameters of the old bridge structure is the key issue. The method of state inversion is based on a series of processes such as finite element modeling of bridge structure, parameter detection, model modification, state verification, etc., to realize the noise reduction of measured data samples and real-time dynamic correction of digital model. Generally, the test data cannot directly identify the structure state, and the clear meaning can only be obtained through the structure analysis. Parameter identification with the help of finite element model correction technology has been widely used in structural damage identification and assessment (Yan et al. 2019; Guo et al. 2021).

The mid-span deflection and structural cracking of long-span prestressed concrete bridges are significant. In order to obtain the real state of the structure, the existing appearance diseases (span middle and lower flexion and typical structural cracks) of the bridge structure are verified with the results of the finite element model by state inversion, and the structural parameters and material properties that cannot be determined by the bridge structure detection are deduced by combining the measured data of the bridge structure. In order to obtain the true state of the structure, the influence of the existing appearance defects (mid-span deflection and typical structural cracks) of the bridge structure is brought into the finite element model for analysis through the state inversion method, and checked with the measured data of the bridge to deduce the structural parameters and material properties that cannot be determined by the bridge structure detection.

3.1.1 Identification parameter selection

Since there is a correlation between the structure parameters, the structure identification parameter selection must be treated separately according to its degree of correlation. For prestressed concrete girder bridges, structural stiffness damage and prestress loss cannot be quantified and analyzed by traditional detection methods. Therefore, the bending stiffness and effective prestress of the main beam are taken as the identification parameters. In order to simplify the analysis, other factors are removed during the identification of beam damage and effective prestress inversion. For the bending stiffness damage of the main beam, the load test results over the years are used for analysis. For the loss of prestress, the cumulative deflection value of the main beam (excluding other factors) is used for inverse identification.

At present, the detection of the effective prestress of the internal beam of bridge structure is mostly a semi-damage detection method, and the detection sample of bridge is limited and the measurement accuracy is difficult to guarantee, so the assessment of the real internal force state of the bridge often has a large deviation. In order to predict and control the construction status of bridge demolition on the basis of reasonable internal force state of dead load, it is necessary to accurately evaluate the effective prestress condition of the bridge.

3.1.2 Stiffness damage identification based on verification coefficient of deflection

According to the crack distribution characteristics of the main beam, the distribution of various types of cracks is relatively uniform, so the overall stiffness reduction coefficient is used to equivalently simulate the stiffness loss caused by the cracking of the main beam (Wu et al. 2023). The decrease in structural stiffness will cause changes in deflection and self-resonance frequency, so the structural stiffness change is generally determined by bridge load test. The following will analyze and evaluate the actual situation of the bridge structure based on the deflection check coefficient and frequency in the monitoring report over the years, and comprehensively judge the cross-sectional damage of the box girder.

1) First, establish the initial finite element model according to the relevant information of the drawing; 2) Through the load testing in the construction stage and current stage, the deflection and frequency data before and after the damage of the bridge structure are obtained, respectively; 3) Adjust the reduction coefficient by comparing the deviation between the equivalent coefficient and the structural verification coefficient.

The deflection verification coefficient obtained based on the static load test can reflect the stiffness of the bridge. Define the deflection verification coefficient as follows:

$$\xi_1 = \frac{S_1}{S_{d1}} \tag{1}$$

Where S_1 is the measured deflection value under the action of the test loading; S_{d1} is the theoretical calculated deflection value under test loading; ξ_1 is the ratio of the measured value to the theoretical value.

In order to determine the stiffness damage of the structure, the modified model is used for static load test loading. The formula for calculating the equivalent coefficient t is:

$$\xi_2 = \frac{S_2}{S_{d2}} \tag{2}$$

Where S_2 is the measured deflection value under test loading (structural damage); S_{d2} is the theoretical calculated deflection value under test loading (modified model).

The measured deflection value of the load testing in the construction completion stage is calculated according to Eq. (1) to obtain the structural verification coefficient ξ_1 in the initial state. According to the relevant parameters of the load testing before demolition, the stiffness reduction coefficient is adjusted to obtain the deflection value of the modified model, and the equivalent coefficient ξ_2 of the bridge structure before demolition is calculated according to Eq. (2). According to the deviation of the equivalent coefficient and the structural verification coefficient, the accuracy of the stiffness reduction coefficient is verified.

3.1.3 Stiffness damage identification based on verification coefficient of frequency

As an inherent parameter of the bridge structure, frequency can be comprehensively evaluated based on the verification coefficient of frequency. From the theory of mechanics, the relationship between structural stiffness and frequency can be obtained as shown in Eq. (3):

$$\omega_n = (\frac{n\pi}{l})^2\sqrt{\frac{EI}{m}} \tag{3}$$

Where ω_n is structural frequency; n is frequency order; m is the quality of the structure. Define β as the verification coefficient of frequency, shown in Eqs. (3 and 4):

$$\beta = \frac{\omega_d}{\omega_s} \tag{4}$$

Where ω_d is theoretical calculation frequency; ω_s is measured frequency.

3.1.4 Identification of prestress loss

In the design of long-span prestressed concrete bridges, the total prestress loss is usually superimposed after the itemized calculation of each prestress loss. However, in the actual construction process, due to the influence of tension process, material properties, operating environment and other factors, it is difficult to accurately calculate the total prestress loss. The estimation of prestress loss is too high, which may increase the risk of local failure of the concrete at the anchor end or cracking in the tensile zone. If the prestress loss estimation is low, the crack resistance and vertical stiffness of the structure cannot be effectively improved. Through the statistical analysis of the total prestress loss value, relevant domestic scholars pointed out that the prestress loss of concrete beams can reach more than 30% of the tensile control stress (Zhang and Liu 2002; Zhu 2010; Zhang et al. 2018; Liu et al. 2019). How to accurately evaluate the degree of prestress loss of prestressed tendons has become the key to the accurate assessment of bridge demolition construction status.

In order to comprehensively grasp the effective pre-stress size of the bridge pre-stress system, a refined simulation model is mainly used to invert the mid span deflection state, estimate the effectiveness evaluation of the entire bridge pre-stress, and verify the detection parameters of the bridge structure to ensure the reliability of the model parameters. When analyzing, it is necessary to comprehensively consider factors such as uneven settlement of the foundation, linear temperature difference during measurement, and long-term shrinkage and creep of concrete. The principle of prestressed inversion identification is as follows (Kernicky et al. 2018; Li et al. 2022) (Fig. 11).

3.2 Structural safety assessment method based on impact matrix

3.2.1 Structural safety evaluation index

According to the second section parameter error influence analysis, the typical ultimate failure mode of continuous rigid bridge under the influence of the most unfavorable parameter combination is the strength failure caused by excessive cross-section stress level, so the structural safety evaluation mainly takes the stress water at the limit failure

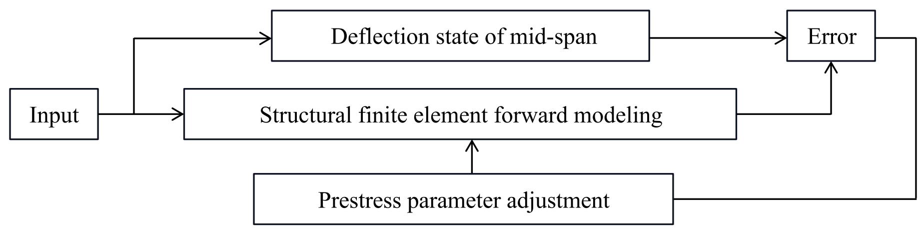

Fig. 11 Inversion identification of prestress loss degree

section position as the index, and its safety limit is the material strength standard value of the corresponding structure. When there is an error in the strength of the material, the corrected data is used as the evaluation standard.

3.2.2 Structural safety assessment principle based on impact matrix

In bridge demolition construction, some parameters are generally tested by relevant detection technology, and then the calculation model is adjusted to obtain stress results to determine construction safety, which is cumbersome and cannot quickly determine structural safety. Based on the identification results of the parameter envelope on the most unfavorable section position of the dismantling section of the continuous rigid bridge, the safety assessment is carried out by combining the influence matrix and finite element calculation. The structural safety assessment at the construction stage can be carried out with the help of the influence matrix. According to the basic principle of the influence matrix, the structural stress influence formula is defined as follows (Tian et al. 2015; Zhang et al. 2013; Cho et al. 2021; Liu et al. 2022a, b):

$$\{\sigma\} = [W]\{A\} \tag{5}$$

Where $\{\sigma\}$ is the structural vector of stress increment, It is specifically expressed as $\{\sigma\} = \{\sigma_1,\ \sigma_2, \cdots \sigma_m\}^T$; $\{A\}$ is the column vector composed of structural influence parameters, expressed as $\{A\} = \{a_1, a_2, \cdots a_n\}^T$; $[W]$ is the stress increment influence matrix ($n \times m$ order), it represents the incremental change of cross-section stress caused by the change of structural influencing parameters.

The method of obtaining the influence matrix vector is as follows: adjust the value of the parameter unit percentage error, extract the cross-section stress result in the corresponding construction stage, and calculate the cross-section stress increment. During construction, according to the measured or inverted results of each parameter, the corrected parameter value can be brought into the calculation to obtain the actual stress state of the structure, so as to carry out structural safety assessment.

3.2.3 Structural safety assessment process

The specific assessment process of structural safety is:

1) Confirm the limit failure mode and location of the structure through the influence of parameter error;

2) The damage parameters are identified by check coefficient and inversion identification method;
3) The specific stress state of the limit failure section position is calculated through the above influence matrix;
4) Confirm the safety of the structure through the comparison of material strength levels;
5) Finally, the correctness of the theoretical analysis calculation is verified by the actual bridge test.

4 Construction safety guarantee of actual bridge based on security evaluation method

Similarly, for the above-mentioned prestressed concrete continuous rigid bridge, the beam cross-sectional damage was identified through the completion load test and the pre-demolition load test at the time of completion. Through the downward deflection of the main beam in the long-term operation stage, the prestress loss is inverted and identified. Then, according to the identification results, the safety of the closure section is evaluated, and verified according to the monitoring data analysis.

4.1 Identification of structural damage parameters of actual bridges

4.1.1 Damage identification of beam section

In order to quantify the degree of damage caused by cross-section cracking of the main beam, the measured deflection value obtained by the static load test before demolition and the theoretical calculation value of the correction model are calculated, and the deflection check coefficient of the main beam of the case bridge is calculated as shown in the following Table 4.

According to different stiffness reduction models, the equivalent coefficient is close to the deflection verification coefficient when the stiffness is reduced by 15%, indicating that the overall stiffness of the structure is reduced by about 0.85 times the initial stiffness.

According to the dynamic load testing results, the structural verification coefficient ξ_1 is calculated according to Eq. (4); the frequency value of the modified model is obtained by adjusting the stiffness reduction coefficient, and the equivalent coefficient ξ_2 is

Table 4 Stiffness damage identification results based on verification coefficient of deflection

Test position	Construction completion stage	Pre-demolition stage		Deviation
	Deflection verification coefficient ξ_1	Stiffness reduction	Equivalent coefficient ξ_2	
Mid-span	**0.81**	0% (initial model)	0.96	19%
		-5%	0.90	11%
		-10%	0.84	4%
		-15%	**0.80**	1%
		-20%	0.76	-6%

calculated. The accuracy of the equivalent coefficient is verified by the deviation between the verification coefficient and the equivalent coefficient.

According to the calculation of different stiffness reduction models (Table 5), the equivalent coefficient β_2 when the stiffness is reduced by 10% is close to the measured frequency verification coefficient β_1 of the dynamic load testing, indicating that the overall stiffness of the structure is reduced by about 0.9 times the initial stiffness, which is close to the deflection test result of 0.85 times, indicating the reliability of the calculation, and it can be preliminarily determined that the cross-section damage is reduced by 15%.

4.1.2 Inversion identification of prestress loss

According to the bridge mid-span deflection monitoring values corrected by the temperature effect of the case bridge, it can be seen that the cumulative mid-span deflection of the left side span is 75.5 mm, the cumulative mid-span deflection of the middle span is 62.0 mm; the cumulative mid-span deflection of the right span.

In order to fully grasp the actual effective prestress of the bridge, the inversion of the deflection state of the mid-span is carried out through the refined finite element model, and the effective prestress assessment of the whole bridge is carried out. Comprehensively considering the uneven settlement of the foundation, the measured temperature difference, the long-term shrinkage, creep and prestress loss of concrete, the mid-span deflection of the left and right bridge are compared, as shown in the Table 6.

According to the calculation results (Table 6), the prestress reduction ratio of the left bridge is about 9 ~ 13%, and the reduction ratio of the right bridge is about 10 ~ 19%. In order to facilitate the calculation, the average value is uniformly taken as the basis for subsequent theoretical calculations, that is, the left width is reduced by 11%, and the right width is reduced by 15%. However, the MIDAS calculation results show that the instantaneous loss of tension force is about 10 ~ 15%, so the inversion result is similar to the research of relevant scholars, which can accurately reflect the real constant load internal force state of the bridge.

4.2 Actual bridge security evaluation based on parameter identification results

According to the above analysis, the parameters such as prestress, self-weight of the main beam, and closing temperature difference have a more significant influence on

Table 5 Stiffness damage identification results based on verification coefficient of frequency

Frequency order	Formation	Construction completion stage	Pre-demolition stage		Deviation
		Frequency verification coefficient β_1	Stiffness reduction	Equivalent coefficient β_2	
1st order (fundamental frequency)	Vertical formation (full bridge anti-symmetry, single-span positive symmetry)	**0.97**	0% (initial model)	1.03	7%
			-5%	1.00	3%
			-10%	**0.96**	-1%
			-15%	0.93	-3%
			-20%	0.90	-6%

Table 6 Inversion of long-term mid-span deflection

Number	Influence factors of mid-span deflection	Left bridge		Right bridge	
		Mid-span (mm)	Sid-span (mm)	Mid-span (mm)	Sid-span (mm)
①	Uneven settlement	0.0	0.0	-8.0	1.2
②	Shrinkage and creep	-26.2	-29.0	-24.9	-29.6
③	Concrete over-square	-6.4	-11.3	-1.9	-12.3
④	Stiffness reduction	-17.4	-17.4	-17.4	-17.4
①+②+③+④	Total	-67.3	-75.0	-69.5	-75.4
⑤	Measured value (temperature correction)	-62.0	-75.5	-78.2	-71.0
⑤-(①+②+③+④)	Prestress loss	-12.0	-17.8	-26.0	-12.9
Inversion results of prestress loss		**-8.8%**	**-13.1%**	**-19.2%**	**-9.5%**

the force of the structure, so the background bridge was analyzed by relevant detection means before the demolition, and the theoretical volume increased by about 8.1% by 3D laser scanning. Through the historical completion data, it is known that the closing time is about October 96, and after checking the historical weather, compare the temperature with the demolition construction stage, and take the temperature difference of -6 °C in the demolition stage; Through the gouging detection of the prestressed part of the pores, the grouting is not full accounting for 41.7%, and 87% of the prestress has different degrees of rust, so the steel bar elastic mold reduction parameter is taken (-20%×87%), the grouting incomplete part is reduced according to 50% anchoring effect, and the prestressed anchorage degree parameter is taken (100%-41.7%×50%); The effective prestress, concrete mold and other detection methods have large errors, low data reliability, and cross-section cracking reduction is difficult to quantify, so they are analyzed with the most unfavorable parameters, and the specific value results are as follows (Table 7):

From the above analysis, it can be seen that the most unfavorable sectional positions of the structure in the demolition stage of the mid-span closure segment are the mid-span near the cantilever end (section #1), the top of pier (section #2) and the variable section (section #3) of the side-span side, and the structural stress changes under different parameter combinations were calculated according to the influence law of seven parameters: self-weight, moment of inertia, effective prestress, anchorage degree, elastic modulus of concrete, elastic modulus of prestressed tendon and construction temperature difference. Obviously,

$$\{\sigma\} = \{\sigma_1, \sigma_2, \sigma_3\}^T, \{A\} = \{a_1, a_2, \cdots a_7\}^T$$

Specifically, the calculation result of the impact matrix is as follows (when the positive and negative effects of parameter errors are different, the bracketed values indicate that the parameter error is positive):

Table 7 Value of background bridge security assessment parameters

Parameters	Value	Reason	Remarks
Self-weight	8.1%	Structural volume detection	Detection value
Moment of inertia	-15%	The most unfavorable value degree	Identification value
Effective prestress	-15%	The most unfavorable value	Identification value
Anchoring degree-prestress	79.2%	Inspection and assessment of duct	Test and assessment value
Modulus of Elasticity-C	-20%	grouting quality	Assessment value
Modulus of Elasticity-Ps	-17.4%	The most unfavorable value	Test and assessment value
structural strength	-10%	performance degradation	Assessment value
Temperature difference	-6℃/20℃	Rust detection	Detection value

Table 8 Security assessment results of background bridge (MPa)

Section position	Theoretical calculation of stress	Stress increment of parameter correction	Corrected stress	Structural strength (correction)	Conclusion
Section #1	-1.36	-15.87	-17.23	-19.48	Safety
Section #2	-1.21	-1.96	-3.17	-15.73	
Section #3	-0.94	2.03	1.09	1.61	

$$[W] = \begin{bmatrix} 1.47 & 3.89 & -6.12(-1.47) & 14.22 & 0 & 0.33(-0.07) & -0.07 \\ 1.82 & 0 & -2.55(-1.32) & -0.02 & 0 & 0.29(0.17) & -2.99 \\ 0.55 & 0 & 3.89(0.38) & -0.40 & 0 & -0.33(0.03) & 3.26 \end{bmatrix} \times 1\%$$

The value of each parameter is brought into the influence parameter vector to obtain $\{A\} = \{8.06, -15, -15, 79.15, -20, -17.4, -30\}^T$, and the stress increment is further obtained $\{\sigma\} = \{-15.87, -1.96, 2.03\}^T$ (Table 8).

The main girder of the case bridge is made of C48 concrete, and the piers are made of C38 concrete. After reducing the design value of concrete strength by 10%, the allowable compressive stress of the main beam is $f_{cd-L}^{'} = -19.48MPa$, the allowable compressive stress of the pier is $f_{cd-D}^{'} = -15.73MPa$, and the allowable tensile stress is $f_{td-D}^{'} = 1.61MPa$, and the stress of the most unfavorable section position can be judged to meet the requirements, so the safety risk of demolition construction of the closure segment of the bridge is low.

4.3 Construction safety control technology based on structural stress-displacement monitoring

The above theoretical calculation and analysis show the structural safety of the demolition stage of the closure segment. In order to further verify the correctness of the above theoretical calculation results, ensure the safety of demolition construction, and clarify the mechanical behavior characteristics of the bridge demolition process. With the help of stress-displacement automatic monitoring technology, the section stress and displacement monitoring are controlled during the demolition stage of the closure segment. The stress measurement points of the main beam are mainly arranged in the 0# cantilever root and section at 1/4 span (mid-span bottom plate bundle range, choose the 4# segment) to arrange the stress measurement points; Displacement measurement points are arranged at the segment line of each main beam. The stress measurement points of the pier are mainly arranged at the bottom of the pier and the variable section; Displacement measurement points are arranged at the top of the pier (Fig. 12).

4.3.1 Structural response of girder in demolition stage

After the demolition construction of the mid-span closure segment, the vertical displacement of the girder is as follows (for example, LD1-1 represents the measurement point of the 1# segment on the small mileage side, and LD1'-1 represents the measurement point of the 1# segment on the large mileage side) (Fig. 13, Table 9).

In general, the measured response of the displacement of the main beam in the demolition stage of the closure segment is basically consistent with the theoretical calculation results. It can be seen from Fig. 14. That the measured stress change of the main beam in the demolition stage of the mid-span closure segment is consistent with the overall theoretical calculation. (Example: LS1-01~LS1-03 represents the stress measurement point of the top slab of the 0# segment on the small mileage side, and LS1-04~LS1-05represents the stress measurement points of the bottom slab of the 0# segment on the small mileage side; LS1'-4X represents the stress measurement point of the 4# segment on the large mileage side).

4.3.2 Structural response of pier in demolition stage

During the demolition stage of the middle span closure segment, the horizontal displacement and stress changes of the main pier are shown in the figure below. It can be seen from the figure that the trend of measured displacement and stress is basically consistent with the theoretical calculation, and the measured result is slightly smaller than the theoretical calculation, but the overall difference is not large (DS1-1~DS1-4 are stress measuring points at variable cross-section, and DS1-5~DS1-8 are stress measuring points at pier bottom (Figs. 15 and 16).

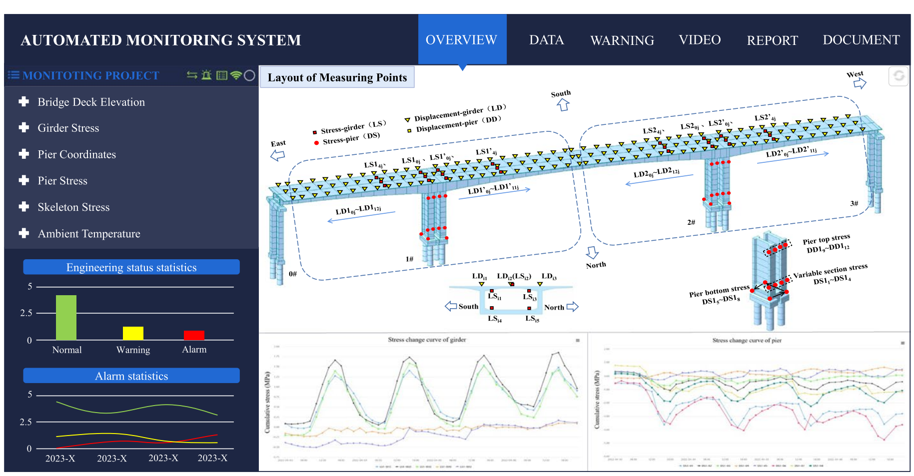

Fig. 12 Schematic diagram of automatic monitoring system

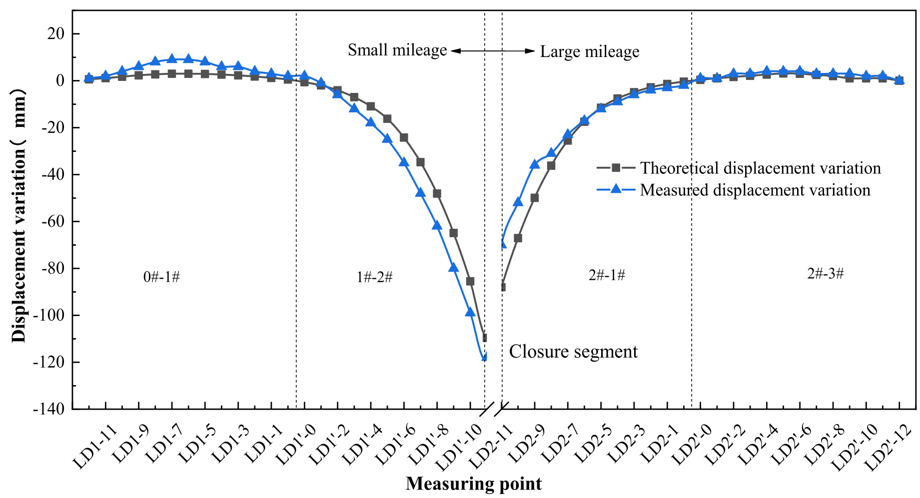

Fig. 13 Vertical displacement variation diagram of girder during the demolition of mid-span closure segment

Table 9 Maximum vertical displacement of girder (mm)

Position	Measuring point	Theoretical calculation	Measured results	Error
Closure segment - small mileage side	LD1'-11	-112.1	-118.3	5.5%
Closure segment - small mileage side	LD2-11	-74.1	-75.3	1.6%

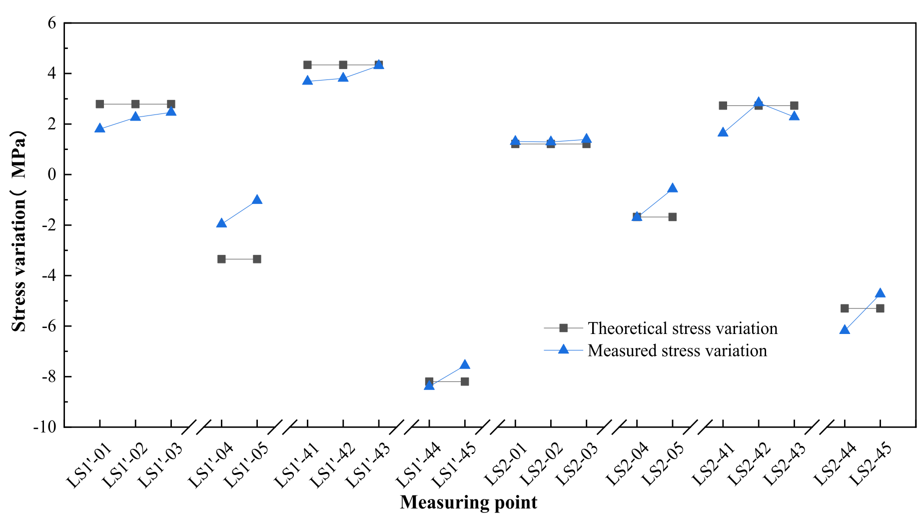

Fig. 14 Stress variation diagram of girder during the demolition of mid-span closure segment

Through the comparison of on-site monitoring data and theoretical calculation results, it can be seen that the parameter value based on structural detection and inversion identification has high accuracy, and also shows the correctness of the

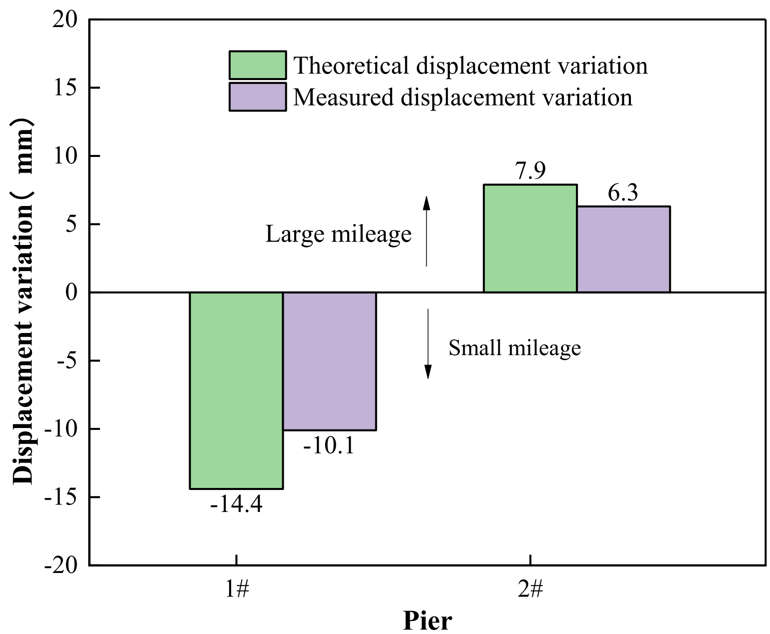

Fig. 15 Horizontal displacement variation diagram of pier

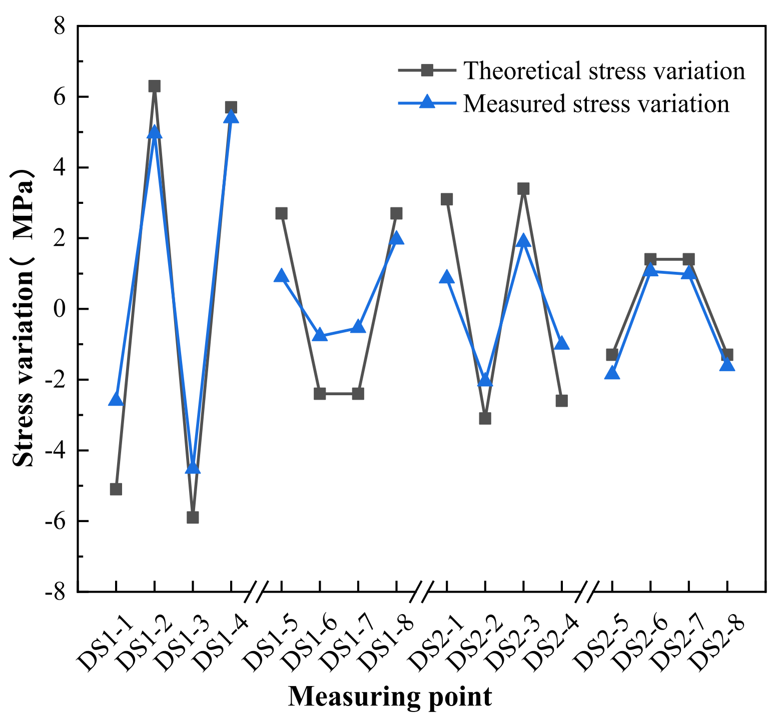

Fig. 16 Stress variation diagram of pier

theoretical method of structural safety assessment, which can be used to guide similar bridge demolition construction.

5 Conclusion

In order to clarify the construction risks of demolition of long-span continuous rigid bridges and propose a reasonable safety assessment method to ensure the safety of demolition construction of the closure segment, this paper analyzes the structure stress state of the demolished construction of the closure segment of the long-span continuous rigid bridge by combining finite element analysis, theoretical calculation and actual measurement verification. According to the analysis results, a structural safety assessment method based on structural calculation identification and influence matrix is proposed, and finally a specific engineering case is taken as an example, the parameter damage identification and safety assessment analysis is carried out, and the correctness of the analysis is verified according to its on-site monitoring results. The main findings are as follows:

(1) Based on the main error source and error value range of the calculation parameters of the continuous rigid bridge, the influence of the parameters in the demolition stage of the closure segment is analyzed through the finite element analysis platform, and it is pointed out that the parameters that have a greater impact on the force of the main beam and pier are mainly the self-weight of the structure, effective prestress, and construction temperature difference of the closure segment.

(2) Through envelope analysis of the influence of various parameters, it was determined that the ultimate failure mode during the dismantling stage of the middle span closure segment of the continuous rigid frame bridge under the most unfavorable parameter combination is the collapse of the mid-span at bottom plate of girder due to excessive compressive stress, and the cracking of the bridge pier top and variable cross-section due to tensile stress exceeding the limit.

(3) In order to further accurately evaluate the safety of demolition construction of the closure segment, a structural safety assessment method based on structural damage identification and influence matrix is proposed. That is, the cross-sectional damage of the beam body is identified by the verification coefficient, the prestress loss is inverted and identified by the long-term downflex state inversion technology of the main beam, and the stress influence results under different parameter values are calculated through the influence matrix of stress and calculation parameters. Then, according to the identification results and the stress influence matrix, the stress level of the most unfavorable section is calculated, and the structural safety state is evaluated according to the material strength criterion.

(4) Through case analysis, the safety of the demolition of the middle span closure segment of the background bridge is clarified, and the demolition process of the closure segment of the Actual Bridge is monitored with the help of automatic monitoring technology, and the rationality of the theoretical analysis is verified by the analysis of measured data.

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Authors' contributions
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Availability of data and materials
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Declarations

Competing interests
The authors declare that they have no competing interest.

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