

ADVANCED STEEL CONSTRUCTION

An International Journal

Volume 22 Number 3

June 2026

CONTENTS

Technical Papers

Fatigue Crack Recognition and Quantification under Complex Background Based on Region Detection
Yu-Hang Liu, Zhi-Yuan Yuan Zhou, Bo-Hai Ji, Fei-Ya Rong and Tian Gao

Measurement Point Optimization Method for Large-Span Space Structures Based on Structural Response Correlation
Ren-Zhang Yan, Wei Liu, Yang Liu, Chun-Ling Yan and Tao Zhu

Hysteretic Behavior of H-Section Aluminum Alloy Members under Axial Loading: Experimental and Numerical Study
Xiao-Nong Guo, Yin Zou and Xing-Ye Wang

Study on Post-Fire Mechanical Performance of Q460qenh Weathering Steel with Three Cooling Methods
Bing-Sen Wang, Hang Yin, Li-Juan Zheng, Er-Nian Zhao and Kai Yan

Theoretical Feasibility Study of the New Steel Grating Web Composite Beam
Jun Hu, Yong-Yong Liang, Xin-Qi Zhang, Fu-Ming Yang and Liang-Jian Lai

Uncertainty Quantification of Steel Truss Assembly Accuracy Based on Error Propagation and Metamodel-Based Methods
Li-Hang Chen, Dong Liang, Hai-Bin Huang, Li-Chao Su and Bo Wang

Study on Shear Test and Calculation Method of Corrosion Cold-Formed Thin-Walled Steel with Self-Drilling Screw-Pull Rivet Hybrid Connection
Yun-Peng Chu, Ming-Bo Huang, Xue-Qin Chen and Jin-Rong Fu

Finite Element Analysis on Hysteretic Performance of Hollow Ribbed High-Strength Concrete Filled Square Steel Tubular Columns
Ying Xie and Xiao-Chuan Chen

Research on the External Restrained Units Stiffness of Segmented Assembled Buckling-Restrained Braces
Shuai Xu, Cheng-Xin Guo, Yan Li, Xiu-Li Wang, Mei-Qiu Zhan and Yuan-Hong Hu

Damage Control and Optimal Design of Embedded CFRP Circular Tube CFSST Column-H-Beam Joint with Energy-Dissipation Angle Steel
Guo-Chang Li, Yong Ma, Xiao Li, Wei-Qi Cui, Zhi-Bin Han and Zhuang-Wen Qin

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Website: <http://www.hkisc.org>

ISSN 1816-112X

Science Citation Index Expanded, Materials Science Citation Index and ISI Alerting

Cover: Guangzhou Baiyun International Airport T3, China. Photo@ Chen Hui-Min

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ADVANCED STEEL CONSTRUCTION

VOL.22, NO.3 (2026)

ADVANCED STEEL CONSTRUCTION

an International Journal

ISSN 1816-112X

Volume 22 Number 3

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ISSN 1816-112X

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The International Journal of Advanced Steel Construction provides a platform for the publication and rapid dissemination of original and up-to-date research and technological developments in steel construction, design and analysis. Scope of research papers published in this journal includes but is not limited to theoretical and experimental research on elements, assemblages, systems, material, design philosophy and codification, standards, fabrication, projects of innovative nature and computer techniques. The journal is specifically tailored to channel the exchange of technological know-how between researchers and practitioners. Contributions from all aspects related to the recent developments of advanced steel construction are welcome.

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Advanced Steel Construction

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- Fatigue Crack Recognition and Quantification under Complex Background Based on Region Detection 255
Yu-Hang Liu, Zhi-Yuan Yuan Zhou, Bo-Hai Ji, Fei-Ya Rong and Tian Gao*
- Measurement Point Optimization Method for Large-Span Space Structures Based on Structural Response Correlation 263
Ren-Zhang Yan, Wei Liu, Yang Liu, Chun-Ling Yan and Tao Zhu*
- Hysteretic Behavior of H-Section Aluminum Alloy Members under Axial Loading: Experimental and Numerical Study 275
*Xiao-Nong Guo, Yin Zou and Xing-Ye Wang**
- Study on Post-Fire Mechanical Performance of Q460qenH Weathering Steel with Three Cooling Methods 286
Bing-Sen Wang, Hang Yin, Li-Juan Zheng, Er-Nian Zhao and Kai Yan*
- Theoretical Feasibility Study of the New Steel Grating Web Composite Beam 298
Jun Hu, Yong-Yong Liang, Xin-Qi Zhang, Fu-Ming Yang and Liang-Jian Lai*
- Uncertainty Quantification of Steel Truss Assembly Accuracy Based on Error Propagation and Metamodel-Based Methods 307
Li-Hang Chen, Dong Liang, Hai-Bin Huang, Li-Chao Su and Bo Wang*
- Study on Shear Test and Calculation Method of Corrosion Cold-Formed Thin-Walled Steel with Self-Drilling Screw-Pull Rivet Hybrid Connection 315
Yun-Peng Chu, Ming-Bo Huang, Xue-Qin Chen and Jin-Rong Fu*
- Finite Element Analysis on Hysteretic Performance of Hollow Ribbed High-Strength Concrete Filled Square Steel Tubular Columns 323
Ying Xie and Xiao-Chuan Chen*
- Research on the External Restrained Units Stiffness of Segmented Assembled Buckling-Restrained Braces 331
Shuai Xu, Cheng-Xin Guo, Yan Li, Xiu-Li Wang, Mei-Qiu Zhan and Yuan-Hong Hu*
- Damage Control and Optimal Design of Embedded CFRP Circular Tube CFSST Column-H-Beam Joint with Energy-Dissipation Angle Steel 344
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FATIGUE CRACK RECOGNITION AND QUANTIFICATION UNDER COMPLEX BACKGROUND BASED ON REGION DETECTION

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ABSTRACT

Accurate measurement of fatigue crack length is essential for routine maintenance of steel bridges, and automated technologies have already been applied in maintenance operations. However, conventional image processing strategies suffer from complex and multi-source interference in steel box girders. This study proposed a two-stage method combining region recognition and morphological processing to diminish noise in fatigue crack images, thereby improving the accuracy of crack recognition and length measurement. The object-recognition-former (OBFormer) model was built to detect and isolate the fatigue crack regions, leveraging the advanced feature extraction module and localization capabilities. Optimized Canny algorithm was utilized to suppress noise and extract the skeletal line of the fatigue crack from detected regions. The length of the crack was calculated by modifying the coordinate system of the crack plane and quantifying the number of pixels along the skeletal line. Experimental results showed that the OBFormer model effectively handled complex background interference and accurately detected the fatigue crack regions, with a mean average precision (mAP) of 91%. After morphological processing, the length of the crack skeleton was accurately calculated with a relative error of less than 5%. The proposed methodology provides a reliable and efficient solution for fatigue crack measurement, offering significant potential for real-world applications in structural health monitoring and bridge maintenance.

ARTICLE HISTORY

Received: 25 March 2025
Revised: 23 July 2025
Accepted: 2 August 2025

KEYWORDS

Steel box girder;
Fatigue crack;
Computer vision;
Deep learning

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1. Introduction

Orthotropic steel decks (OSDs) have been widely used in long-span bridges for their impressive spanning capabilities[1], whereas they are particularly vulnerable to fatigue cracks under the high load traffics[2]. Fatigue cracks could significantly undermining both mechanical performance and durability of bridges[3–5]. As the number of cracks continues to increase, the maintenance pressure correspondingly intensifies. Based on computer vision, automatic defect inspection systems had been developed to replace manual inspection. Particularly, crack detect model based on neural networks had been proved to be applicable to engineering practice, and showed its accuracy and high efficiency[6–8].

In early stage, computer vision tasks focused on objection detection which were applicable to the pavement crack detection challenge and had gotten great results[9]. For example, Chu et al.[10] proposed the Ghost-YOLO model to address the heavy workload associated with crack detection in CNNs, achieving mean average precision of 84.77%. Ye et al.[11] applied the YOLOv7 architecture to concrete damage detection and found that the model demonstrated strong crack detection capabilities in complex environment. With the rapid development of computer vision technology, the focus of crack detection task was moved to semantic segmentation. Semantic segmentation could acquire the accurate geometrical shape through pixel-level classification. This method showed great performance in concrete crack detection[12–15].

Compared to pavement cracks, fatigues cracks in OSDs show undistinct visual characteristics mainly due to the small width. In the meantime, the surfaces of OSDs exhibit noise which shows a resemblance to the visual characteristics of fatigue cracks and reduces the accuracy of neural networks. Therefore, semantic segmentation method showed some problems when it was applied to crack detection on OSDs images. Specifically, some models misclassified small objects with visual features similar to cracks. For example, Dong et al. [16] demonstrated that edges of weld lines might be detected as small fatigue crack fragments by using the proposed pixel-level fatigue crack segmentation method. Xu et al.[17] proposed fusion convolutional neural network, which had a small probability of misclassifying handwritten notes as fatigue cracks.

This study proposes a method for recognizing and quantifying fatigue cracks in OSDs to overcome interference targets while considering model lightweight. OBFormer model is designed and employed to identify cracks regions and to extract localized images. An optimized edge detection algorithm is applied to obtain the crack skeleton. Finally, the number of skeletal pixels is counted to calculate the crack length from distortion-corrected 2d image.

2. Crack detection on 2d image

2.1. Architecture of OBFormer

OBFormer is divided into encoder part and decoder part. Encoder is composed of multiple Transformer blocks. Each block outputs a feature map $Feat_i$, which is passed to next block as well as the decoder. Decoder mainly consists of Up-Down feature fusion (UDFF) module and classification-repression-interaction (CRI) module. In training progress, images are divided into multiple overlapping patches in the Overlap Patch Embedding (OPE) module, and each patch is encoded into a vector representation. After processing through the OPE module, the feature maps are fed into the Transformer block to compute the different-level feature maps. UDFF module adjusts the sizes and channels of deep-level feature maps and then fuse these features. The processed feature maps are transmitted to CRI module to extract discriminative features. All the features from different layer are concatenate to form a comprehensive feature vector to calculate the output.

Transformer block consists of Efficient Self-Attention module (ESA) mechanism, Mix Feedforward Network (Mix-FFN), and Overlapped Patch Merging (OPM) module. The mathematic progress is shown in Fig. 1.

The ESA[18] mechanism is based on the Self-Attention (SA) mechanism[19]. Feature maps are firstly flattened into sequence, and then the Query (Q), Key (K), and Value (V) matrices are computed to establish relationships between any two pixels. These matrices are then passed to the Mix-FFN function to generate the deep-level feature maps. Subsequently, the overlap patch merging (OPM) module reorganizes the metrics back into feature maps. The ESA mechanism is estimated as:

$$\text{Attention}(Q, K, V) = \text{softmax}\left(\frac{QK^T}{\sqrt{d_K}}\right)V \quad (1)$$

where Q , K , and V are tensors describing the queries, keys, and values respectively, and d_K is dimension of K . The Q , K , and V have the same dimensions of $N \times c$, where $N=h \times w$ is the length of the feature sequence. To improve the computational efficiency of the self-attention mechanism, ESA introduces a reduction ratio R to reduce the spatial dimensions of features sequence as follows:

$$\hat{K} = \text{Reshape}(N/R, C \times R)(K) \quad (2)$$

$$K = \text{Linear}(C \cdot R, C)(\hat{K}) \quad (3)$$

Where K is sequence to be reduced, $\text{Reshape}(N/R, C \cdot R)(K)$ refers to reshape K to the one with shape from $N \times C$ to $(N/R) \times (C \cdot R)$, then the new K can obtain with the dimension of $(N/R) \times C$ through a linear transformation. As a result, the complexity of self-attention is reduced from $O(N^2)$ to $O(N^2/R)$.

Positional code is fixed in traditional transformer [20]. When the test resolution is different from that the training one, the positional code needs to be interpolated, which lead to the drop of accuracy. Mix-FFN consider the effect of zero padding to the leak location information by directly using a 3×3 convolution in the feed-forward network (FFN). Mix-FFN is formulated as:

$$\text{Mix-FFN}(x) = \text{MLP}(\text{GELU}(\text{Conv}3 \times 3(\text{MLP}(x)))) + x \quad (4)$$

where x is the feature from the ESA module. Mix-FFN mixes a 3×3 convolution and an MLP into each FFN.

The OPM module is based on Patch Merging (PM) operation in Swin Transformer[21]. The PM module was initially designed to combine non-overlapping image or feature patches, which lacks the ability to preserve continuity between patches. To address this limitation, OPM introduces an overlapped operation on the feature maps. By incorporating overlapping regions between patches, OPM maintains the continuity between patches, thereby improving accuracy of the OBFormer.

The feature maps extracted by the encoder are fed into the UDFE module.

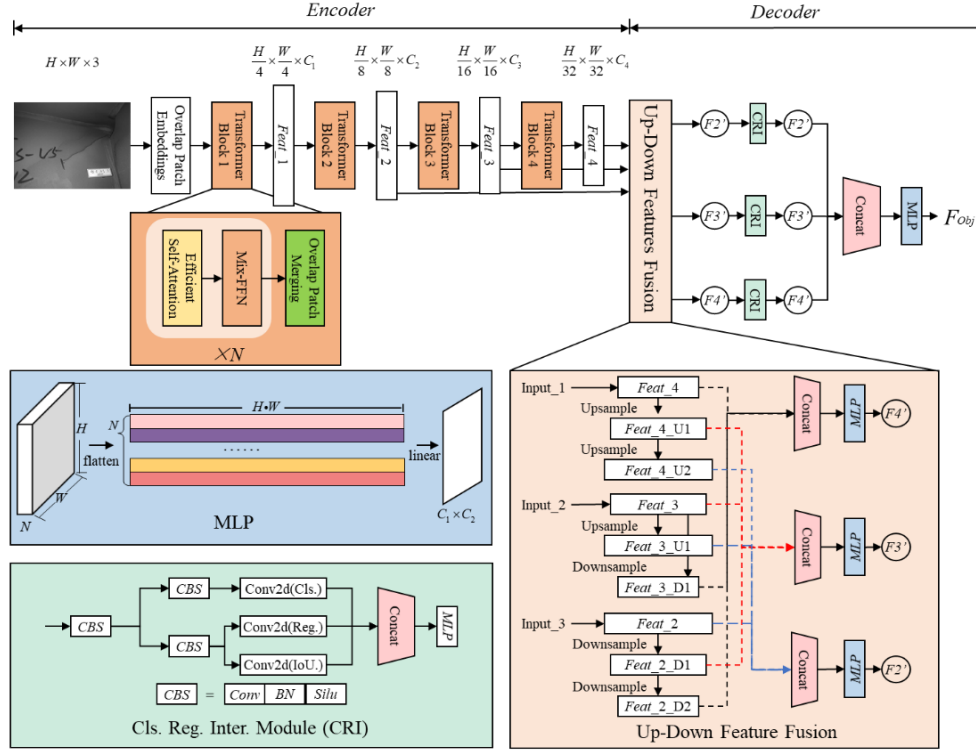


Fig. 1 Structure of OBFormer

2.2. Construction of the dataset

During long-term monitoring and crack detection for two steel bridges, 2,350 images depicting fatigue cracks was collected to create annotated dataset. The images presented several challenges, including issues of overexposure and

underexposure, which were attributed to variations in collection environmental conditions. To address these challenges, the data augmentation techniques were used to enhance the diversity of the data and improve generalization capabilities of OBFormer. The dataset was divided into three subsets in an 8:1:1 ratio for the purposes of model training, validation, and test.

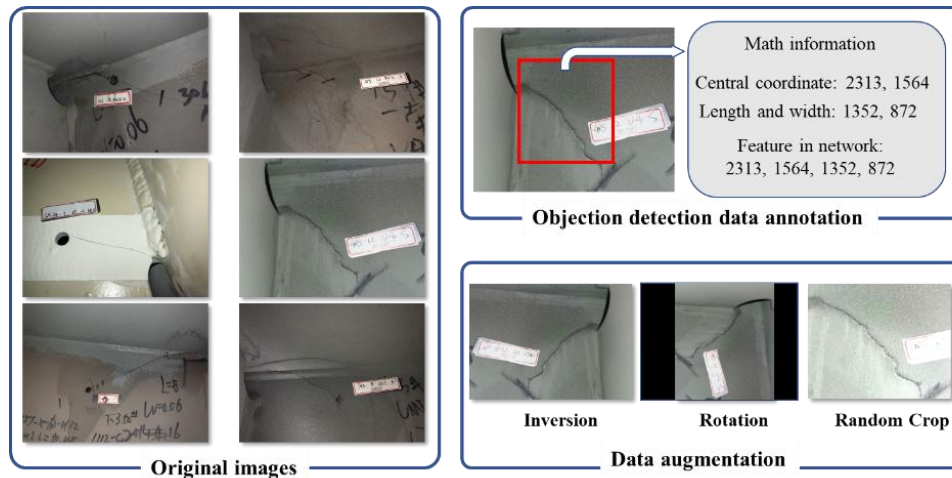


Fig. 2 Dataset construction

2.3. Network training and inference

2.3.1. Training parameters and metrics

In this study, the OBFormer was established and evaluated under the software system of Ubuntu 18.04. The hardware mainly consists of a Ryzen 9-5900X CPU and RTX 3090 GPU with 24GB RAM. The training phase

employed transfer learning strategy to initialize the parameters of the encoder. The initial weights were derived from the training results of the common data set ImageNet. Simultaneously, frozen training strategy was used to optimize training. Learning rate was reduced from $1e-4$ to $1e-7$ through a cosine annealing decay method[22], in conjunction with the AdamW optimizer[23], over 300 training epochs. The hyperparameters are shown in Table 1.

Table 1
Sets of hyperparameter

Hyperparameter	set
Input size	448×448
Batch size	8
Learning rate	$1e-4$ to $1e-7$
Epochs	300
Frozen epochs	20
Optimizer	AdamW

Average precision (AP) was utilized to assess the performance of model. AP measures the ratio that a sample identified as positive is genuinely a true positive, as shown in Eq. (5) as

$$AP = \frac{1}{N} \sum_N TP / (TP + FP) \quad (5)$$

where TP refers to true positive samples, which are target pixels that have been correctly classified. FP denotes false positive samples, which are background pixels erroneously classified as target pixels.

2.3.2. Training results of OBFormer

Training results are shown in the Fig. 3. The loss value curve indicates that both the training and validation value begin to stabilize after the 200th epoch. After the 250th epoch, the training and validation loss values converge to similar levels, indicating good model convergence without signs of underfitting or overfitting. To verify the superiority of OBFormer, two representative state-of-the-art object detection models are selected, YOLOv7 and Mask R-CNN, for comparison. For model training, all hyperparameters are configured as specified in Table 1. The quantitative comparison metric used was mAP, and the training results of the three models are illustrated in Fig. 3 (b). As observed from the curves, all models cease to fluctuate after the 50th epoch, with mAP values steadily increasing thereafter. After the 250th epoch, the values reach a stable region, where OBFormer achieves a final mAP of 0.91, significantly outperforming the other two models. Fig. 3 (c) shows the detection outcomes for various targets within crack images were notably satisfactory. Specifically, the AP values for cracks, stop-holes, and labels were 81%, 95%, and 98%, yielding an overall mAP of 91% across the three categories. Table 2 presents a comparison of OBFormer, YOLOv7, and Mask R-CNN in terms of model size and inference efficiency on the custom dataset. OBFormer has fewer parameters and FLOPs, indicating a more lightweight architecture. Meanwhile, OBFormer achieves an FPS of 126.7 under the same conditions, substantially higher than the other two models, demonstrating its superior real-time processing capability.

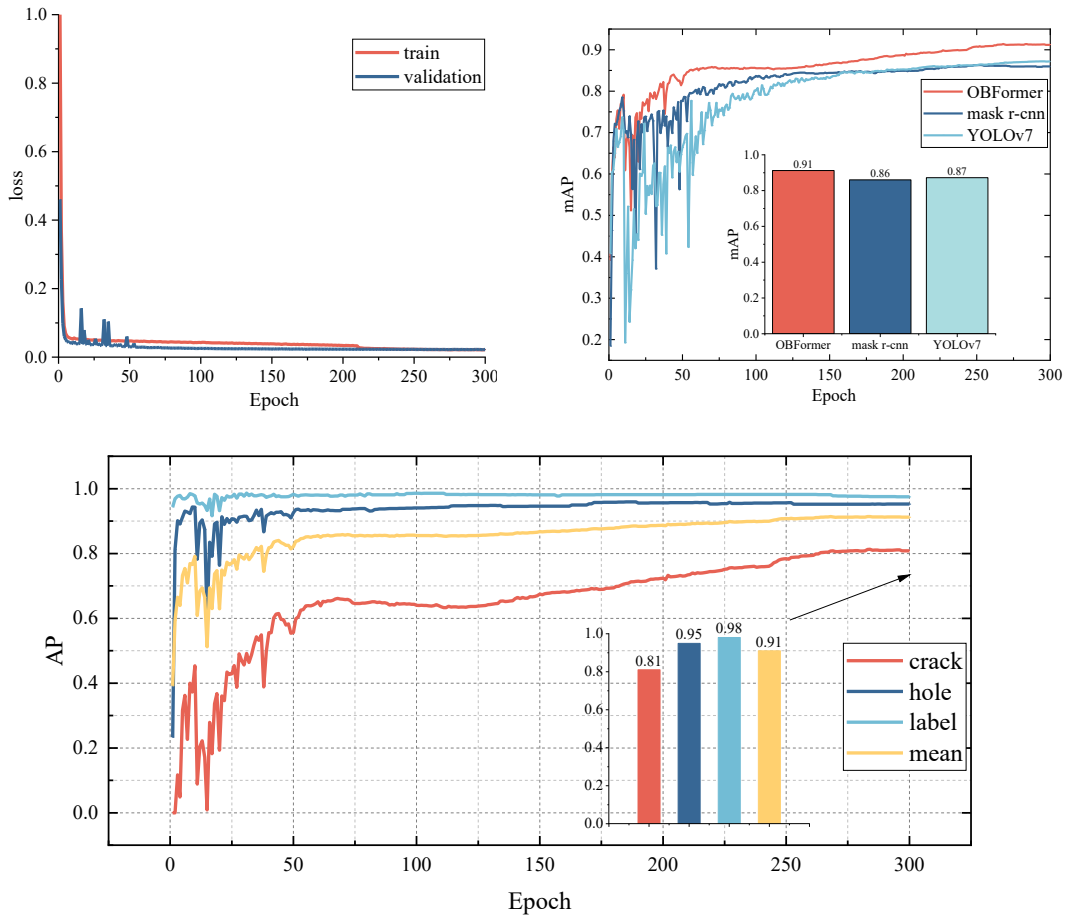


Fig. 3 Training results of OBFormer. (a) Loss curve. (b) Comparison with object detection model. (c) Average precision calculated for all categories

Table 2
Model size

Model	Parameter(M)	Flops(G)	FPS
YOLOv7	37	60	78.3
Mask R-CNN	44	100	35.6
OBFormer	4.53	10	126.7

The deployed results of the model applied to the test dataset are shown in Fig. 4, which shows various typical fatigue cracks, stop-hole and label areas. In instances of single crack, it was practical to reliably estimate the area of the cracks, which serves to reduce the quantity of background pixels. In images containing both stop-holes and multiple cracks, the proposed methodology effectively distinguished and detected the areas associated with each individual crack and stop-holes. This approach avoids the generation of a single prediction

box for multiple cracks. The deployed results indicate that the OBFormer exhibits a strong suitability with the test dataset, demonstrating adequate

generalization capabilities across all data. The OBFormer is capable of accurately predicting the presence of cracks, stop-holes, and labels information.

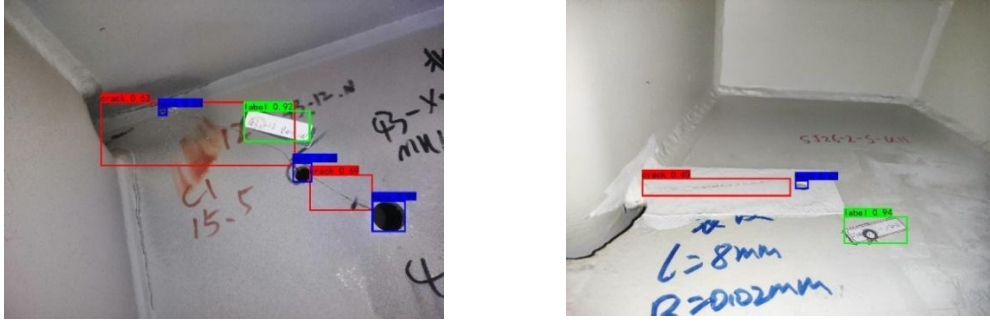


Fig. 4 Results of the detection of representative samples

3. Crack feature extraction

3.1. Optimized edge detection algorithm

The edge detection algorithm presented is derived from the Canny detection algorithm[24], with the Gaussian filtering substituted by bilateral filtering. The bilateral filter incorporates both spatial and range information of pixel points. There is a clear difference in grayscale values between crack and background in image. The diversity enhances the range weight and reduces the spatial weight when convolution kernel is applied to the crack region. The adjustment reduces the smoothing effect on the image, thereby preserving the edge information. In non-crack regions, the filter reduces range weight and increases spatial weight, allowing for smooth processing of the background areas.

Dense bright spots caused by the flash lights result in significant gradients which still exist after smoothing with a large convolution kernel. Based on the characteristics of images collected from steel box girders, where the background is typically dark with low grayscale values and the fatigue crack regions generally have even lower grayscale values, a threshold segmentation preprocessing step is introduced before smoothing the images. All bright spots in the image with grayscale values above the threshold are set to the threshold value by applying an adaptive threshold segmentation method. The mathematical formulation of the algorithm is delineated in Eq. (6)

$$I_{(y)} = \frac{1}{W_q} \sum_{x \in S} I(x) G_s(x, y) G_r(I(x), I(y)) \quad (6)$$

$$G_s(x, y) = \exp\left(-\frac{\|x - y\|^2}{2\sigma_s^2}\right) \quad (7)$$

$$G_r(I(x), I(y)) = \exp\left(-\frac{\|I(x) - I(y)\|^2}{2\sigma_r^2}\right) \quad (8)$$

$$W_p = \sum_{x \in S} G_d \bullet G_r \quad (9)$$

where S represents the set of pixel points within the neighborhood during the filtering procedure. The x and y denote the coordinates of the target pixel point within neighborhood. $I(x)$ indicates the pixel value of each pixel point in the set S , while $I(y)$ represents the pixel value following the application of filtering processes. The G_s represents the geometric space that characterizes pixel values in bilateral filtering. G_r represents the absolute value of the grayscale variation between a designated point and the central point within the neighborhood. Furthermore, W_p represents the sum of the weights of each pixel value within the filtering window, used for weight normalization, expressed as Eqs.(7)-(9). σ_d and σ_r represent the variances of the image and the grayscale space after filtering.

3.2. Results of feature extraction

The deployed results of OBFormer on given images generated parameters related to the predicted bounding box for the target object, which comprises four values: (b_x, b_y, b_w, b_h) . The b_x and b_y denote the coordinates of the center of the bounding box, while b_w and b_h indicate width and height. The pixel position information for the four corners of the bounding box in the original image, represented as $[a, b, c, d]$, can be obtained through Eq. (10).

$$a = b_x - b_h/2, b = b_x + b_h/2, c = b_y - b_w/2, d = b_y + b_w/2 \quad (10)$$

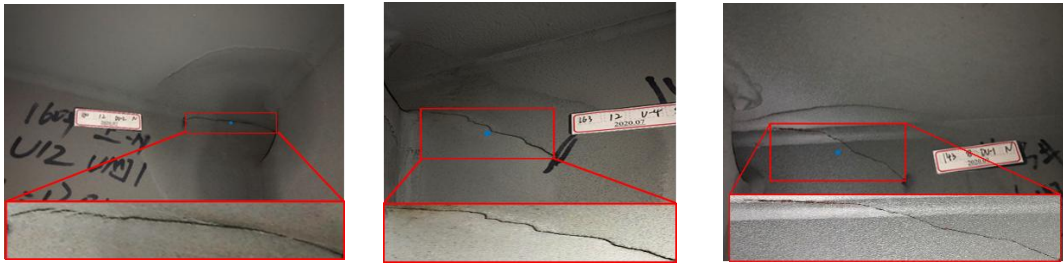


Fig. 5 OBFormer detection results

While OBFormer accurately identifies object-level bounding boxes, precise characterization of structural defects such as cracks requires additional pixel-level edge information. Therefore, an optimized edge detection algorithm was utilized to analyze the images presented in section 3.1, with the results depicted in Fig. 6. The figure illustrates the efficacy of the algorithm in identifying various object edge information within the images, such as cracks, labels, handwritten text, and the contours of over-welded holes. However, noise was present in each image, primarily due to inadequate lighting conditions inside the OSDs. The usage of flashlight during photography, in conjunction with the rough texture of the steel surface, intensified the noise present in the images.

To quantify the denoising performance of the two algorithms, structural similarity (SSIM), peak signal-to-noise ratio (PSNR), and Error Rate were used. Fig. 7 illustrates the results of three representative images processed by two different denoising methods: Gaussian filtering and bilateral filtering. It can be observed that bilateral filtering consistently achieves higher SSIM and PSNR values across all three images, indicating its superior ability to preserve edge features and overall structural information compared to Gaussian filtering. Moreover, the Error Rate is generally lower under bilateral filtering, further confirming its effectiveness in removing noise while minimizing the loss of image details.

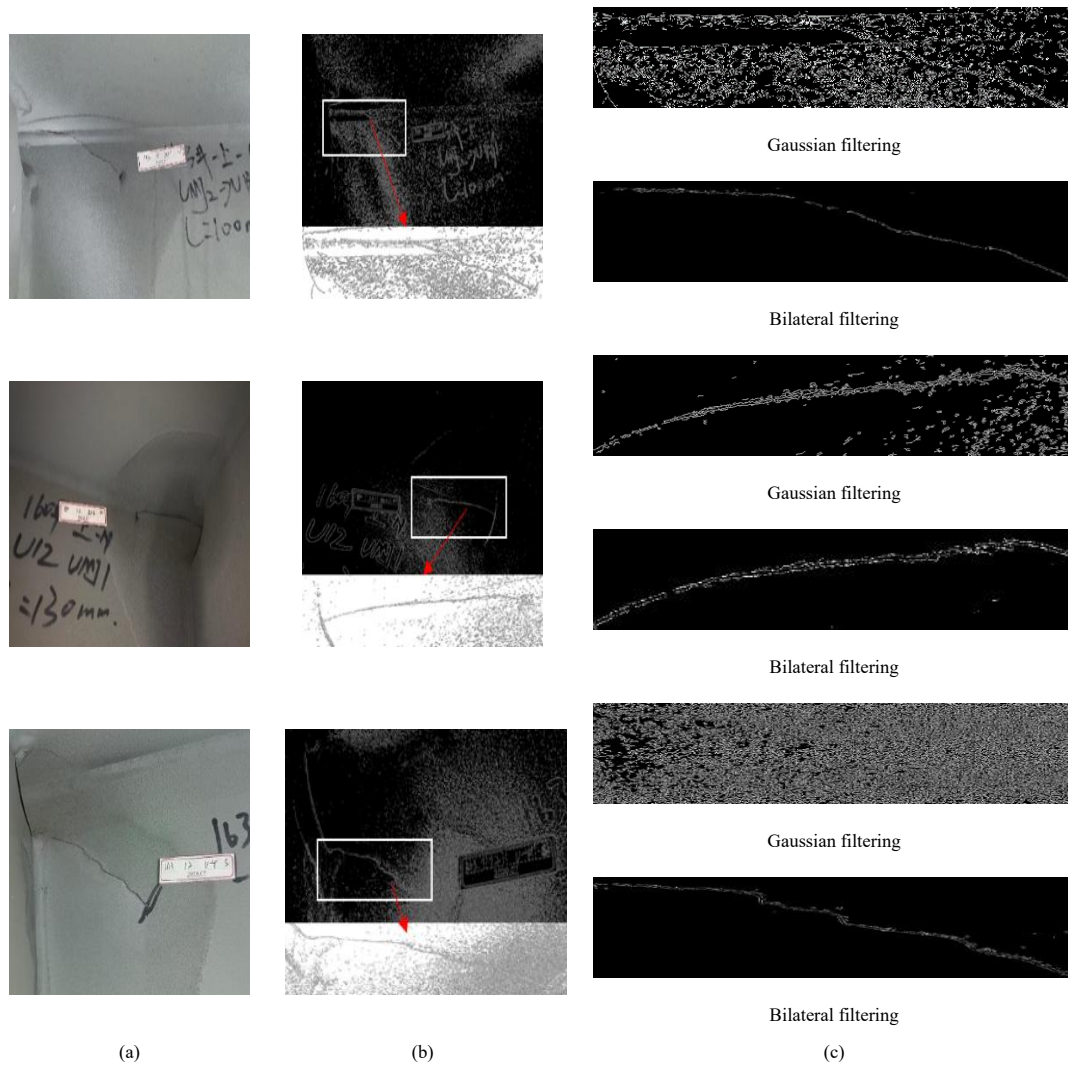


Fig. 6 Results of edge detection. (a) Original image, (b) Whole image, (c) Cropped image

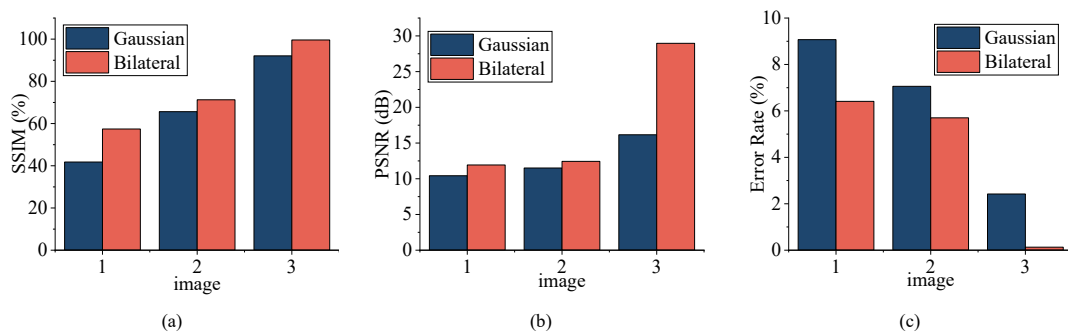


Fig. 7 Comparison of denoising performance between gaussian and bilateral filtering

Bridge maintenance necessitates the collection of data regarding the locations and geometric characteristics of cracks. In the analyzed images, crack regions were manually cropped and repositioned, and color inversion was applied to improve the visibility. Nonetheless, the images were considerably impacted by noise, which obscured the visibility of the crack areas. The presence of text and noise partially disrupted the continuity of the crack paths, complicating the quantification of relevant features. This challenge was exacerbated by the fact that pixels representing the cracks made up only a small proportion of the total pixel count in the images. As a result, the detection process struggled to identify crack areas and corresponding shapes. Therefore, accurately delineating and extracting crack areas from the images was essential for obtaining precise geometric features.

Given that cracks often occupy a small portion of the image and are susceptible to interference from noise and annotation artifacts, crack regions were manually cropped and centered to reduce the influence of surrounding distractors. This preprocessing step also facilitates more effective denoising.

The cropped image was centered on the crack region, reducing the influence of weld-joints, labels, and handwritten notes. However, the image that had undergone Gaussian filtering still contained a significant level of noise, which hindered the accurate delineation of the crack area. The outcomes through the application of bilateral filtering demonstrated that the majority of the noise had been eliminated, resulting in an image that primarily comprises pixel information relevant to the cracks. The edges of the cracks were clearly defined, and the geometric features were emphasized, thereby enhancing the ability to recognize the crack shape.

3.3. Morphological processing

The edge detection results may contain noise or boundary discontinuities making it difficult to directly extract complete crack regions. Therefore, a seed-filling method based on region growing were used to construct connected domains of cracks. The additions enhanced the coherence of the method and

improved the interpretability of the crack extraction process. Crack edges were first detected through feature extraction and then integrated into a connected domain to represent the entire crack area. In this study, a seed-filling method based on the region growing algorithm was employed to establish the connected domain of the crack. Subsequently, a morphological skeletonization technique was implemented, involving an iterative procedure that utilized erosion and opening operations. The iterative process diminished the connected domain, resulting in linear representations while maintaining the topological integrity of the crack shape. The mathematical model is delineated as Eq. (11).

$$S_k(A) = \bigcup_{k=0}^n (A \ominus kB) - (A \ominus kB) \odot (B) \quad (11)$$

where A is the original binary image. B is the convolution kernel used for operations. The symbol $\ominus$ is used to indicate the morphological erosion operation. The symbol $\odot$ represents the morphological opening operation, which is defined as the sequence of erosion followed by dilation. The variable

k indicates the number of iterations applied during the erosion process. Furthermore, n represents the maximum value at which the resultant set $A \ominus B$ becomes an empty set.

The results obtained from the application of the detection and connected domain algorithm to the fatigue crack images are shown in Fig. 8. The images displayed a certain level of point-like noise and discontinuities. To acquire the total crack area, morphological processing was applied to the edge features. A morphological dilation operation was performed on the image, which aided in connecting crack edges in regions. An erosion operation was then used to the dilated image, using a convolution kernel size identical to that of the dilation, to remove noise. The Zhang-Suen algorithm was used to skeletonize the crack, refining the area into lines composed of single-pixel points. The missing segments within the crack curves typically do not exceed 20 pixels, resulting in minimal impact on the overall morphology of the cracks, although they can affect the total pixel count to some extent. To address this, a curve fitting approach was employed to complete the skeleton lines. Compared to the original discontinuities, this method significantly reduces errors in crack pixel quantification.

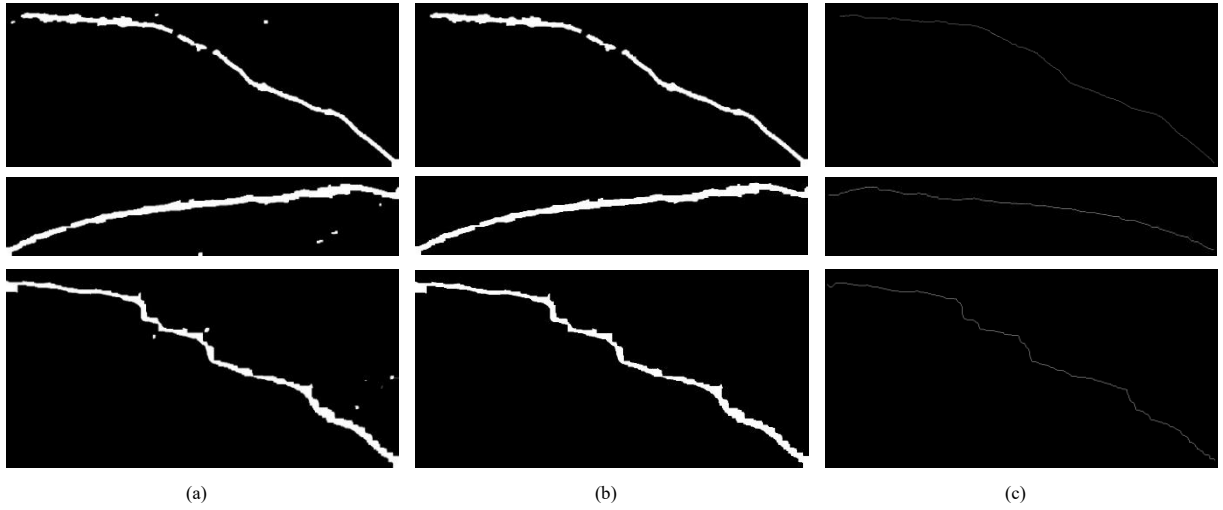


Fig. 8 Results of the morphological processing (a) Enhancement of Canny edge detection outcomes, (b) Results of morphological processing, (c) Crack skeletal lines

4. Feature quantification method

4.1. Monocular theory

In the manual gathering of crack images, a single camera was used for imaging capture. However, achieving a shooting angle that was perpendicular to the crack plane posed significant challenges, thereby complicating the

accurate depiction of crack geometric features within the image plane. The monocular graphical representation is shown in Fig. 9. This study was conducted indoor using a steel box girder specimen with fatigue crack. Given that the gathering of crack images primarily relies on portable cameras in daily maintenance, this study utilized a smartphone as the shooting device. The experimental platform and crack collection process are shown in Fig. 10.

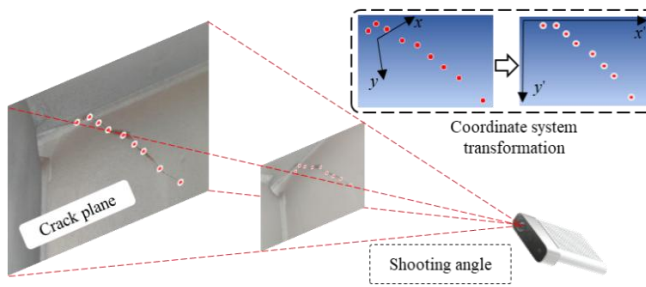


Fig. 9 Method of monocular vision

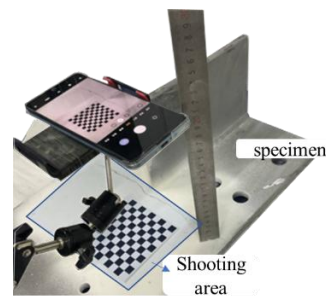


Fig. 10 Experimental platform

The challenge of achieving a shooting angle perpendicular to the crack plane is a key limitation of the monocular vision system. Furthermore, the imaging process may introduce various distortions attributable to factors such as the lens manufacturing processes and the precision of installation. The distortion and intrinsic parameters of the camera are inherent attributes that can be quantified through a procedure known as camera calibration. The mathematical model that describes these parameters is delineated in the following equations:

$$\begin{pmatrix} x' \\ y' \\ 1 \end{pmatrix} = \begin{bmatrix} h_{11} & h_{12} & h_{13} \\ h_{21} & h_{22} & h_{23} \\ h_{31} & h_{32} & 1 \end{bmatrix} \begin{pmatrix} x \\ y \\ 1 \end{pmatrix} = H \begin{pmatrix} x \\ y \\ 1 \end{pmatrix} \quad (12)$$

$$M = \begin{bmatrix} f_x & 0 & u_0 \\ 0 & f_y & v_0 \\ 0 & 0 & 1 \end{bmatrix} \quad (13)$$

$$x' = x(1 + k_1 r^2 + k_2 r^4 + k_3 r^6) + [2p_1 xy + p_2(r^2 + 2x^2)] \quad (14)$$

$$y' = y(1 + k_1 r^2 + k_2 r^4 + k_3 r^6) + [p_1(r^2 + 2y^2) + 2p_2 xy] \quad (15)$$

where H signifies the homography transformation matrix, which articulates the correspondence between points in the pixel coordinate system and those in the world coordinate system. M refers to the specific formulation of the camera intrinsic parameter matrix. The variables x and y denote the coordinates prior to correction, whereas x' and y' indicate the coordinates following correction. The $k_1, k_2, k_3, p_1, p_2, f_x, f_y, u_0$, and v_0 are variables that necessitate estimation. The proposed solution methodology involves initially calculating the intrinsic and extrinsic parameters of the camera through the homography matrix, subsequently employing nonlinear least squares to estimate the distortion parameters. The derived parameters are calculated using the maximum likelihood estimation approach.

4.2. Experiment of feature quantification

Position the chessboard calibration plate, measuring 10 cm by 10 cm, on the surface of the steel box beam specimen. Adjust the camera orientation and capture a series of images of the chessboard calibration plate from various

angles for camera calibration. The parameters of the camera are shown in **Table 3**.

Table 3
the result of parameters calculation

Distortion parameters		Camera parameters	
k_1	0.09093	f_x	775.75
k_2	0.09174	f_y	770.83
k_3	-0.52407	u_0	488.24
p_1	-0.01217	v_0	329.91
p_2	0.00228		

This study employs a standard camera projection model, utilizing the intrinsic camera matrix and pose parameters to perform forward projection from 3D points to 2D image coordinates for reprojection error calculation. The multi-angle corner detection results are shown in **Fig. 11**, with an average reprojection error computed as 0.052 pixels.

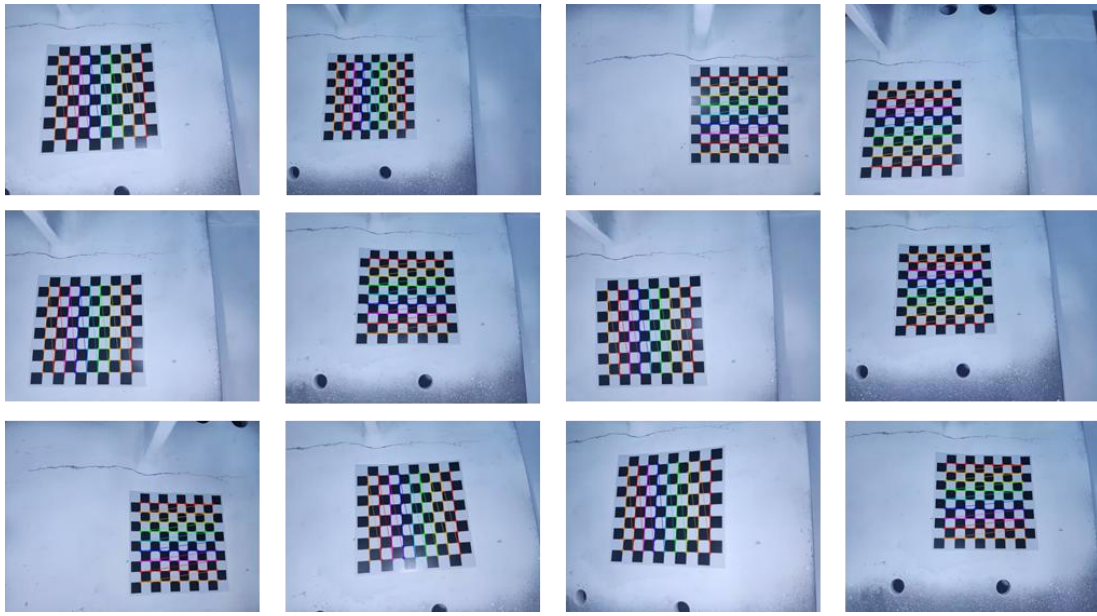
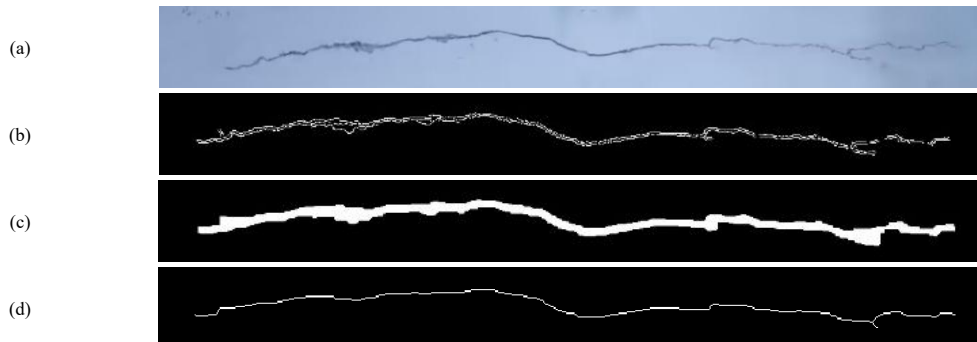


Fig. 11 Chessboard corner detection results

After obtaining the camera intrinsic parameters, feature quantification analysis was conducted on 9 fatigue crack specimen images captured at varying distances and angles. The OBFormer model was employed to detect and crop local images depicting fatigue cracks. The image processing techniques described in Section 3 are applied to the acquired images to identify the crack edges and the associated connected components, following dilation and erosion operations. Subsequently, crack skeletons are extracted from the images of the connected components. The presentative experimental results are shown in **Fig. 12**.

Fig. 12 (e) illustrates the quantified crack length results from nine images captured under varying shooting angles and distances. The angles include $\pm 45^\circ$ and 90° , and the distances are 20 cm, 30 cm, and 40 cm. In the figure, blue bars represent the crack lengths extracted through image-based recognition, while the red line indicates the variation in relative error. As observed, the relative error ranges from a minimum of 1.8% to a maximum of 4.4%, all within 5%. These results demonstrate that the proposed crack skeleton extraction and breakpoint completion algorithm achieves high stability and accuracy under different imaging conditions.



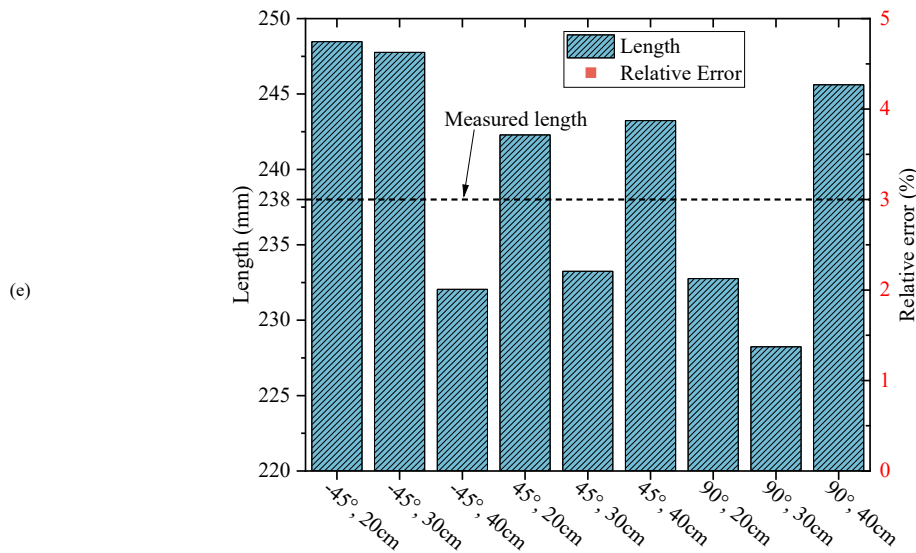


Fig. 12 Results of feature quantification

(a) Cropped results, (b) Optimization algorithm for extracting crack edges,
(c) Crack connected component, (d) Crack skeletal line, (e) Quantification results

5. Conclusion

Recognizing and quantifying of fatigue cracks in steel box girders is highly relevant for daily maintenance, as it impacts the maintenance strategies. The primary objective of this study is to establish a methodology for quantifying crack length through the implementation of area recognition algorithms. Therefore, the study utilizes the OBFormer object detection model alongside daily maintenance image data to calculate the crack length. The efficacy of this approach has been confirmed.

From the previous research, the following conclusions were made:

1. The OBFormer model exhibits high effectiveness in detecting cracks while overcoming image interference on surfaces of OSDs, resulting in mAP of 91.2%.
2. The truncation method for threshold segmentation, combined with an optimized Canny edge detection algorithm, effectively extracts fatigue crack edges under various interference conditions. Additionally, when employed alongside post-processing algorithms, this approach facilitates the accurate extraction of the crack skeletal line.
3. The method for quantifying fatigue crack length in steel box girders, integrated with monocular vision technology, allows for the precise measurement of fatigue crack length, typically achieves a relative error of less than 5%.

Acknowledgment

This work was supported by National Natural Science Foundation of China [grant numbers 52308166, 52378153].

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MEASUREMENT POINT OPTIMIZATION METHOD FOR LARGE-SPAN SPACE STRUCTURES BASED ON STRUCTURAL RESPONSE CORRELATION

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ABSTRACT

The mechanical properties of large-span space structures are complicated and change greatly during its construction. So it is necessary to monitor the real time status of the structure to ensure the safety of the construction process. However, the number of monitoring points is often very large due to the space structure's large number of members and three-dimensional forcing characteristics. An optimization method for reducing monitoring points is proposed based on the characteristics of certain correlation between different members' mechanical responses during construction in this paper. Firstly, the comprehensive correlation matrix of structural response was established by using correlation coefficient method and grey correlation degree method, and the cluster correlation matrix with block characteristics was calculated by the bond energy algorithm, so as to construct the classification and optimization principle of reducing measuring points. Then, LSTM neural network was used to build a response prediction model, and the in-situ monitoring data was used to train and verify the prediction model. The results show that Pearson correlation coefficient and B-type grey correlation can effectively explore the similarity between the change size and trend of spatial structural responses. At the same time, LSTM neural network can learn to optimize the response correlation between the measuring points and its related measuring points, realizing the global monitoring of the structure.

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ARTICLE HISTORY

Received: 20 February 2025
Revised: 4 August 2025
Accepted: 18 August 2025

KEYWORDS

Large-span space structure;
Response correlation;
Measurement point optimization;
Neural network

1. Introduction

Due to their large scale and the fact that large-span space structures usually undergo multiple structural system transformations during the construction process, the construction process often becomes the most dangerous stage in their entire life cycle. Therefore, in order to ensure the construction safety of large-span space structures, sensors need to be deployed on key members of the structure to obtain the real-time status of the structure. However, there are a lot of rods in large-span space structures, and they are in a three-dimensional spatial forcing state. There are also many measuring points, which greatly increases the cost and maintenance difficulty of the entire monitoring system. At the same time, more measurement points also increase the amount of data collection, which leads to a large number of data redundancy and increases the cost of later data analysis and storage. To this end, scholars at home and abroad have conducted a series of studies on the optimization of measurement points. For example, both Peng[1] and Lu[2] used the Pearson correlation coefficients to measure the correlation between the responses of the measurement points, established a correlation matrix of structural response, and achieved better arrangement of structural measuring points; Zhu[3] also adopted the Pearson correlation coefficient to calculate and analyze the correlation between multi-strain sensors, and establish a correlation model of the measured data with the multiple linear regression algorithm and the hybrid particle swarm algorithm to achieve the optimization goal of measurement points. In addition, some scholars have also proposed methods to establish correlation prediction models based on neural networks with the application of deep learning in the field of structural damage identification. In addition, Sahar also builds an improved genetic algorithm through the geometric viewpoint and the numerical forward algorithm of Genetic algorithm (GVGA), and is used to optimize the placement of sensors in the structure, and gets a good effect[4]. For example, Feng[5] proposed a correlation modeling method for the nonlinear behavior of bridge structures with threshold recurrent units of recurrent neural networks; Kaveh A employed backpropagation and radial basis function neural networks to train efficient models for the analysis, design, and prediction of dome displacements. [6]. Yang[7] deeply analyzed the correlation characteristics of measurement points in sensors and proposed a recognition method of joint neural network structural state model based on Convolutional Neural Network (CNN) and Gated Recurrent Neural Network (GRU), which provides the basis for optimizing predictions from sensor data.

Through the above research results, it is not difficult to find that the correlation coefficient proposed by most scholars is used to describe the linear relationship between variables. However, potential correlation between variables in actual engineering is far more than linear correlation, and linear measurement indicators cannot satisfy the changes in internal forces of complex structures. Such is the case with large-span space structures. As we know, a large-span space structure has obvious nonlinear properties in the construction

process: On the one hand, the space structure will experience a complex time course during its construction, during which it will not only experience various construction loads and construction error factors, but also undergo multiple structural system transformations. Its geometry and boundary conditions show strong time-varying characteristics, so it presents nonlinear characteristics in time series; On the other hand, when the span of a large-span space structure increases, the stiffness decreases and the geometric nonlinearity is obvious. The correlation between the mechanical response of individual members may also show nonlinearity. For example, literature [5] pointed out that the influence of geometric nonlinearity should be considered in the mechanical analysis of construction process of a large-span space structure. When analyzing the construction internal forces of a large-span lattice shell structure, literature [7] found that the nonlinearity had a great influence on the construction internal forces of the members during the construction process. So it is easy to see that the large-span space grid structure has a relatively obvious nonlinear effect in the construction process, that is, the internal force of each member will change nonlinearly with time, loading history and system transformation, and the change law of each member may be different, it is not difficult to predict that the relationship between the internal force of each member may also be nonlinear. At the same time, in the aspect of structural state recognition and prediction, the time-dependent relationship of response monitoring data is often ignored.

To this end, this paper relies on an actual project of a large-span space grid structure, based on the characteristics of certain correlation between structural responses. First, the nonlinear correlation between the stress responses of different members was found by combining the correlation coefficient method and grey correlation method. Then, the appropriate measuring points were selected based on the bond energy algorithm. Finally, the LSTM neural network was used to predict the mechanical response of the relevant measurement points. It is worth nothing that the measurement points optimization method proposed in this paper can not only obtain the nonlinear correlation between the responses of the large-span space grid structure with mathematical method, but also track the entire construction process when predicting the responses of relevant members, and consider the dependence of the correlation between members in the time dimension.

2. Project overview

The central roof of Mall of China adopts an orthogonal truss grid structure in the form of free-form surface. The north-south span of the structure is 139m~108m, and the east-west span is 231m~165m; the height of the grid in the middle region is 5m, and that near the supports is 4~6m. The grid size is 11m×11m. The web members and chord members of the grid adopt spatially intersecting joints, and hollow spherical joints are used to connect all chord members and between chord members and diagonal braces.

In order to improve construction efficiency, the structure adopts a mixed construction method that combines partitioned assembly, cumulative lifting and high-altitude bulk installation. During construction, the central lifting area is divided into three areas, which are cumulatively lifted after ground assembly. The specific construction steps are: after the ground assembly of the first area is completed, it is raised to the height of the second area and connected with the second area that is installed in bulk on the temporary support frames. Then the first and second areas are raised to a certain height as a whole and connected with the third area that is installed in bulk on the support frames. Finally, the three areas are together raised to the designed height as a whole, and spliced with the surrounding hoisting areas at a high altitude. The structure construction process and the overall diagram are shown in Fig.1. Combined with the stress characteristics during the structural construction process, it can be divided into four stages: In the first stage, the support frame in the second area is unloaded, and the first and second areas are lifted as a whole; In the second stage, the support frame in the third area is unloaded, and the first, second and third areas are lifted as a whole; In the third stage, the support frame in the inner and outer rings of the lifting area is unloaded; In the fourth stage, the lifting point is unloaded.

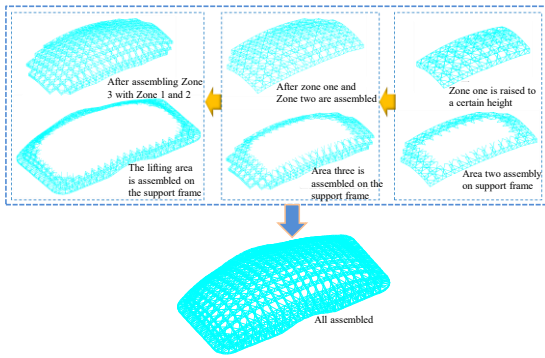


Fig. 1 Central roof steel structure construction process diagram

3. Measurement point optimization method based on response correlation

3.1. Calculation method of response correlation considering multiple construction stages

There are usually two methods to obtain the correlation between the response of the members. One is to find the mechanical relationship between the members within the structure from the mechanical equilibrium and compatibility conditions based on the configuration characteristics of the structure. For the long-span spatial grid structure, due to the large number of member and the high redundancy of the structure, more additional constraints are required to solve the problem. The calculation process is impossible to complete, and in more cases, infinite group solutions will be solved. The other is the use of mathematical statistics, through machine learning sample data, to find out the correlation between different members, its calculation process does not directly depend on the internal mechanical relations of the structure, with operability.

Based on the above reasons, according to the relative change magnitude and trend of structural responses in the construction process, this paper adopts currently commonly used the correlation coefficient method and grey correlation degree to describe the correlation degree between responses and the correlation degree of response change trend over time. In order to measure the similarity of variation and trend between structural responses in the construction process, correlation coefficient method and grey correlation method are used to generate response comprehensive correlation matrix. The calculation process is as follows:

3.1.1. Correlation coefficient method

The correlation coefficient is an indicator that describes the degree of correlation between variables. The Pearson correlation coefficient is commonly used in engineering to measure the similarity between responses. Its calculation principle is:

It is assumed that a structure is divided into α construction stages, and m measuring points are arranged on this structure. If the structural response vector obtained by the i -th ($i=1, 2, \dots, m$) measuring point within n time steps of the k -th ($k=1, 2, \dots, \alpha$) construction stage is $x_i = [x_i(1), x_i(2), \dots, x_i(n)]^T$, then the structural response matrix of m measuring points at n time steps in the k th construction stage can be obtained, as shown in formula (1).

$$X = \begin{bmatrix} x_1 & x_2 & \dots & x_i & \dots & x_m \\ x_1(1) & x_2(1) & \dots & x_i(1) & \dots & x_m(1) \\ x_1(2) & x_2(2) & \dots & x_i(2) & \dots & x_m(2) \\ \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \\ x_1(t) & x_2(t) & \dots & x_i(t) & \dots & x_m(t) \\ \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \\ x_1(n) & x_2(n) & \dots & x_i(n) & \dots & x_m(n) \end{bmatrix} \quad (1)$$

In the formula, the Pearson correlation coefficient of the response vectors x_i and x_j of any two measuring points i and j in the k th construction stage can be calculated according to formula (2).

$$p_{ij} = \frac{\text{cov}(x_i, x_j)}{\sqrt{\sigma_i \sigma_j}} = \frac{E(x_i - \bar{x}_i)(x_j - \bar{x}_j)}{\sqrt{\sigma_i \sigma_j}} \quad (2)$$

Among them, σ_i and σ_j are the variances of the responses at measuring points i and j respectively, which is calculated according to formulas (3) and (4); $\bar{x}_i$ and $\bar{x}_j$ are the response averages of measuring points i and j within n time steps respectively; $\text{cov}(x_i, x_j)$ is the covariance of the responses of measuring points i and j , which is calculated according to formula (5).

$$\sigma_i = \frac{1}{n-1} \sum_{t=1}^n (x_i(t) - \bar{x}_i)^2 \quad (3)$$

$$\sigma_j = \frac{1}{n-1} \sum_{t=1}^n (x_j(t) - \bar{x}_j)^2 \quad (4)$$

$$\text{cov}(x_i, x_j) = \frac{1}{n-1} \sum_{t=1}^n (x_i(t) - \bar{x}_i)(x_j(t) - \bar{x}_j) \quad (5)$$

According to formulas (2) to (5), the correlation matrix P of the structural response change of m measuring points at the k th construction stage is obtained, as shown in formula (6).

$$P = [p_{ij}]_{m \times m} = \begin{bmatrix} p_{11} & p_{12} & \dots & p_{1j} & \dots & p_{1m} \\ p_{21} & p_{22} & \dots & p_{2j} & \dots & p_{2m} \\ \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \\ p_{i1} & p_{i2} & \dots & p_{ij} & \dots & p_{im} \\ \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \\ p_{m1} & p_{m2} & \dots & p_{mj} & \dots & p_{mm} \end{bmatrix} \quad (6)$$

The range of Pearson correlation coefficient is $-1 \leq p_{ij} \leq 1$. If $|p_{ij}|$ is closer to 1, the correlation between measurement points i and j is stronger; If $|p_{ij}|$ is closer to 0, it means that the correlation between measuring points i and j is weaker.

3.1.2. Gray correlation degree

The gray correlation degree describes the relative changes between variables during the development process of the system. If the relative change trends of the two variables during the development process are basically the same, the gray correlation degree is large. On the contrary, if the gray correlation degree is small, it indicates that the relative change trend of the two variables in the development process is inconsistent. This paper adopts B-type gray correlation to measure the similarity of change trends between responses, thereby establishing a correlation matrix of structural response change trends. The calculation principle is:

For any two measuring points i and j in the structure, the responses at the k th construction stage are x_i and x_j respectively, and the calculation of their B-type gray correlation degree is as shown in formula (7).

$$r_{ij} = \frac{1}{1 + \frac{1}{n} d_{ij}^{(0)}(t) + \frac{1}{n-1} d_{ij}^{(1)}(t) + \frac{1}{n-2} d_{ij}^{(2)}(t)} \quad (7)$$

$$d_{ij}^{(0)}(t) = \sum_{i=1}^n |x_i(t) - x_j(t)| \quad (8)$$

$$d_{ij}^{(1)}(t) = \sum_{t=1}^{n-1} |x_i(t+1) - x_j(t+1) - x_i(t) + x_j(t)| \quad (9)$$

$$d_{ij}^{(2)}(t) = \sum_{t=1}^{n-2} \left| \begin{aligned} &x_i(t+1) - x_j(t+1) \\ &-2[x_i(t) - x_j(t)] + [x_i(t-1) - x_j(t-1)] \end{aligned} \right| \quad (10)$$

In the formula, $d_{ij}^{(0)}(t)$ is the overall displacement difference of the response at measuring points i and j ; $d_{ij}^{(1)}(t)$ is the overall first-order slope difference of the response of measuring points i and j ; $d_{ij}^{(2)}(t)$ is the overall second-order slope difference of the response of measuring points i and j , which is calculated according to formulas (7)~(10) respectively.

According to the above method, the correlation matrix R of the response change trend of m measuring points in the k th construction stage can be calculated as:

$$\mathbf{R} = [r_{ij}]_{m \times m} = \begin{bmatrix} r_{11} & r_{12} & \cdots & r_{1j} & \cdots & r_{1m} \\ r_{21} & r_{22} & \cdots & r_{2j} & \cdots & r_{2m} \\ \vdots & \vdots & \ddots & \vdots & \vdots & \vdots \\ r_{i1} & r_{i2} & \cdots & r_{ij} & \cdots & r_{im} \\ \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ r_{m1} & r_{m2} & \cdots & r_{mj} & \cdots & r_{mm} \end{bmatrix} \quad (11)$$

B-type gray correlation degree $r_{ij} \in [0,1]$, if the value of r_{ij} is closer to 1, it means that the correlation between measuring points i and j is greater, and the change trends of the two are more similar; if the value of r_{ij} is closer to 0, indicating that the correlation between measuring points i and j is smaller.

3.1.3. Comprehensive correlation matrix of response

In order to measure the similarity of change magnitude and change trend between structural responses in the construction process, the response comprehensive correlation matrix is generated by combining the change magnitude correlation matrix with the change trend correlation matrix. In order to obtain the response comprehensive correlation matrix effectively, the correlation matrix P and R are treated by setting the correlation threshold, and the equivalent correlation matrix with only 0 and 1 coefficients after binarization is obtained. The specific process is as follows:

First, the similarity threshold λ of the response change magnitude is set, and the correlation matrix P' of the equivalent response changes is obtained as:

$$\mathbf{P}' = [p'_{ij}]_{m \times m} = \begin{bmatrix} p'_{11} & p'_{12} & \cdots & p'_{1j} & \cdots & p'_{1m} \\ p'_{21} & p'_{22} & \cdots & p'_{2j} & \cdots & p'_{2m} \\ \vdots & \vdots & \ddots & \vdots & \vdots & \vdots \\ p'_{i1} & p'_{i2} & \cdots & p'_{ij} & \cdots & p'_{im} \\ \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ p'_{m1} & p'_{m2} & \cdots & p'_{mj} & \cdots & p'_{mm} \end{bmatrix} \quad (12)$$

Among them,

$$p'_{ij} = \begin{cases} 1 & |p_{ij}| \geq \lambda \\ 0 & |p_{ij}| < \lambda \end{cases} \quad (13)$$

Then, the similarity threshold μ of the response change trend is set, and the correlation matrix R' of the equivalent response change trends is obtained as:

$$\mathbf{R}' = [r'_{ij}]_{m \times m} = \begin{bmatrix} r'_{11} & r'_{12} & \cdots & r'_{1j} & \cdots & r'_{1m} \\ r'_{21} & r'_{22} & \cdots & r'_{2j} & \cdots & r'_{2m} \\ \vdots & \vdots & \ddots & \vdots & \vdots & \vdots \\ r'_{i1} & r'_{i2} & \cdots & r'_{ij} & \cdots & r'_{im} \\ \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ r'_{m1} & r'_{m2} & \cdots & r'_{mj} & \cdots & r'_{mm} \end{bmatrix} \quad (14)$$

Among them,

$$r'_{ij} = \begin{cases} 1 & |r_{ij}| \geq \mu \\ 0 & |r_{ij}| < \mu \end{cases} \quad (15)$$

According to formulas (14) and (15), the comprehensive correlation matrix

of response $\mathbf{C}^{(k)}$ of m measuring points in the k th construction stage can be written as:

$$\mathbf{C}^{(k)} = [c_{ij}^{(k)}]_{m \times m} = \begin{bmatrix} c_{11}^{(k)} & c_{12}^{(k)} & \cdots & c_{1j}^{(k)} & \cdots & c_{1m}^{(k)} \\ c_{21}^{(k)} & c_{22}^{(k)} & \cdots & c_{2j}^{(k)} & \cdots & c_{2m}^{(k)} \\ \vdots & \vdots & \ddots & \vdots & \vdots & \vdots \\ c_{i1}^{(k)} & c_{i2}^{(k)} & \cdots & c_{ij}^{(k)} & \cdots & c_{im}^{(k)} \\ \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ c_{m1}^{(k)} & c_{m2}^{(k)} & \cdots & c_{mj}^{(k)} & \cdots & c_{mm}^{(k)} \end{bmatrix} \quad (16)$$

Among them,

$$c_{ij}^{(k)} = \begin{cases} 1 & (p'_{ij} + r'_{ij})/2 = 1 \\ 0 & (p'_{ij} + r'_{ij})/2 \neq 1 \end{cases} \quad (17)$$

In the formula, $c_{ij}^{(k)}$ is the comprehensive correlation coefficient of response of measuring points i and j in the k th construction stage, The coefficient is a Boolean matrix containing only 0 and 1, This coefficient combines the Pearson correlation coefficient and the B-type gray correlation degree. When $c_{ij}^{(k)}=1$, it means that the responses of measuring points i and j in the k th construction stage have strong similarity in the overall time series and change trend; When $c_{ij}^{(k)}=0$, it means that the similarity is weak.

By extracting the response data of m measuring points in each construction stage, correlation analysis was carried out, and the response comprehensive correlation matrix $\mathbf{C}^{(1)} \sim \mathbf{C}^{(\alpha)}$ of α construction stages could be obtained by calculating according to equation (16). The comprehensive correlation matrix $\mathbf{C}$ of structural response in all α construction stages could be further obtained as follows:

$$\mathbf{C} = \begin{bmatrix} c_{11} & c_{12} & \cdots & c_{1j} & \cdots & c_{1m} \\ c_{21} & c_{22} & \cdots & c_{2j} & \cdots & c_{2m} \\ \vdots & \vdots & \ddots & \vdots & \vdots & \vdots \\ c_{i1} & c_{i2} & \cdots & c_{ij} & \cdots & c_{im} \\ \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ c_{m1} & c_{m2} & \cdots & c_{mj} & \cdots & c_{mm} \end{bmatrix} \quad (18)$$

Among them,

$$c_{ij} = \frac{c_{ij}^{(1)} + c_{ij}^{(2)} + \cdots + c_{ij}^{(k)} + \cdots + c_{ij}^{(\alpha)}}{\alpha} \quad (19)$$

In the formula, c_{ij} is the comprehensive correlation coefficient of response of measuring points i and j in all α construction stages, $c_{ij} \in [0,1]$. When $c_{ij}=1$, the responses of measuring points i and j have strong similarity in the overall time series and change trends of all construction stages, that is, the structural response at the two measuring points has a strong correlation under all construction stages; When $0 < c_{ij} < 1$, it means that the structural response of the two measuring points only has a strong correlation in certain construction stages; When $c_{ij}=0$, it means that the correlation between the responses of the two measuring points in each construction stage is weak.

3.2. Classification and optimization of measuring points based on bond energy algorithm

According to the comprehensive correlation matrix of response $\mathbf{C}$, the correlation of any two measuring points in the entire construction process can be obtained. However, when there are many measuring points, there are many elements in $\mathbf{C}$, which makes it difficult to classify the measuring points. Therefore, The Bond Energy Algorithm is introduced to perform cluster analysis on $\mathbf{C}$, so that the elements with $c_{ij}=1$ in $\mathbf{C}$ are gathered near the diagonal of the matrix, forming a cluster correlation matrix with block characteristics to facilitate the classification of measurement points.

In order to perform effective row-column transformation on the matrix $\mathbf{C}$, the matrix $\mathbf{C}$ is binary processed again before cluster analysis, and converted into a comprehensive correlation matrix of equivalent response $\mathbf{C}'$ with only two elements, 0 and 1, according to formula (20). Then through cluster analysis, all elements in $\mathbf{C}'$ are finally restored to their original values.

$$c'_{ij} = \begin{cases} 1 & c_{ij} = 1 \\ 0 & c_{ij} \neq 1 \end{cases} \quad (20)$$

The Bond Energy Algorithm (BEA)[8–11] is a permutation-based algorithm that aggregates similar elements within a matrix through row and column rearrangement as well as segmentation operations, with its core objective being to address the automatic grouping of similar elements in high-dimensional correlation matrices, and its computational procedure is illustrated in Fig. 2. BEA takes the equivalent response comprehensive correlation matrix as input and defines the interaction strength between each matrix element and its adjacent elements as "bond energy," and by iteratively adjusting the row and column ordering, it continuously maximizes the global total bond energy (ME); essentially, ME represents the cumulative sum of the bond energies of all elements and reflects the degree of clustering among similar elements within the matrix, and when ME reaches its maximum value, highly correlated elements in the matrix tend to aggregate into diagonal block structures, with each block corresponding to a group of measurement points exhibiting similar response patterns, and the mathematical formulation is given by Equation (21), which accumulates the bond energies between each element and its adjacent elements and employs a coefficient of 1/2 to correct for the double-counting of bidirectional interactions, thereby ensuring that the matrix structure corresponding to the maximum ME directly reflects the "response-similar measurement point groups" in practical engineering applications and achieving a close integration between the algorithmic mechanism and the requirements of structural monitoring.

$$ME = \frac{1}{2} \sum_{i=1}^m \sum_{j=1}^m c'_{i,j} \times (c'_{i,j-1} + c'_{i,j+1} + c'_{i-1,j} + c'_{i+1,j}) \quad (21)$$

In the formula,

$c'_{i,j-1}$, $c'_{i,j+1}$, $c'_{i-1,j}$, and $c'_{i+1,j}$: the left, right, upper, and lower adjacent elements of the element $c'_{i,j}$, corresponding to the correlation coefficients of adjacent measuring points. $c'_{i,0} = c'_{i,m+1} = c'_{0,j} = c'_{m+1,j} = 0$.

The traditional hierarchical clustering requires constructing an $m \times m$ distance matrix and iteratively merging clusters, with each merging operation involving a traversal of the matrix to find the optimal pair, leading to an overall time complexity of $O(m^3)$. In contrast, BEA achieves $O(m^2)$ complexity. This quadratic vs. cubic difference yields a computational gap of m -fold, directly demonstrating BEA's superiority in large-scale scenarios. As the number of data points m increases, this exponential efficiency advantage solidifies BEA's

applicability to high-dimensional engineering datasets.

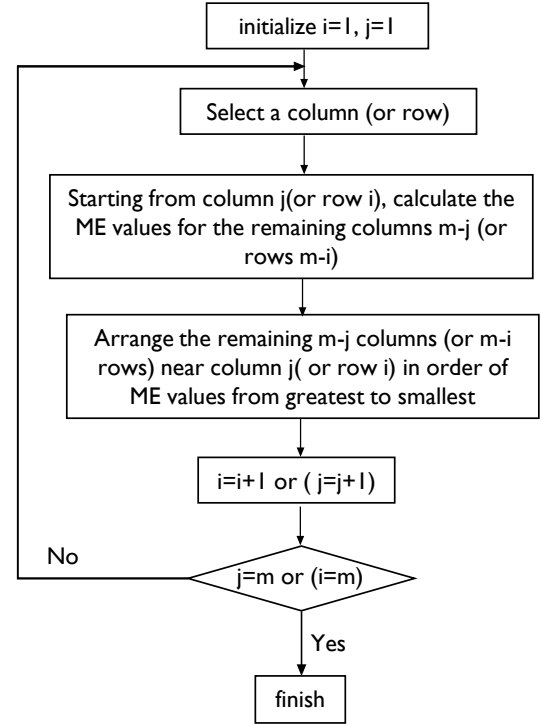


Fig. 2 Flow chart of bond energy algorithm

The comprehensive correlation matrix of response shown in Fig. 3 is taken as an example to illustrate the calculation process of the bond energy algorithm. A total of 10 measurement points from A1 to A10 are shown in Fig. 3. The comprehensive correlation matrix of equivalent response is obtained through binary processing based on the comprehensive correlation matrix of response. Then the original values of all coefficients are restored after transformation by the bond energy algorithm, and finally a clustering correlation matrix with block characteristics is obtained.

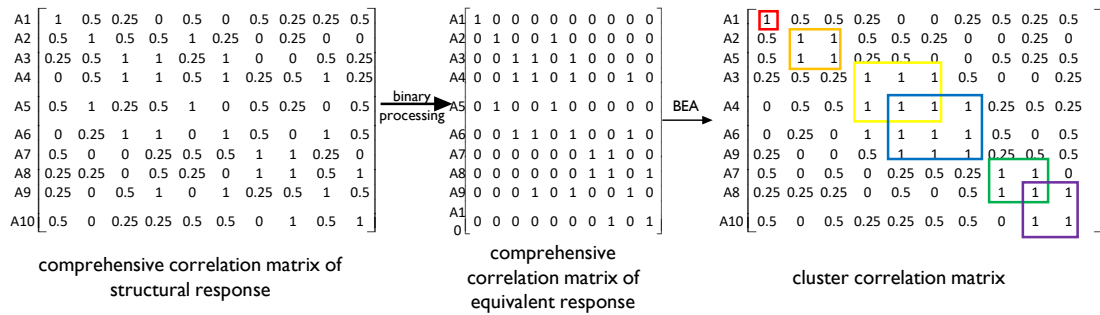


Fig. 3 Correlation matrix transformation diagram

According to the arrangement of each block sub-matrix in the clustering correlation matrix, the measuring points at each location can be further divided into five types[12]: ① If the block sub-matrix contains only 1 measuring point, then the measuring point is of the first type, such as the measuring point A1 shown in Fig. 3; ② If the overlapping parts of two or more adjacent block sub-matrices contains only one measuring point, then the measuring point is of the second type, such as the measuring point A8; ③ If the overlapping parts of two or more adjacent block sub-matrices contain multiple measuring points, then these measuring points are of the third type, such as the measuring points A4 and A6; ④ If the independent block sub-matrix that has no overlapping parts with the adjacent block sub-matrix contains multiple measuring points, then such measuring points are of the fourth type, such as the measuring points A2 and A5; ⑤ The remaining measuring points belong to the fifth type, such as the measuring points A3, A7, A9 and A10.

For the first type of measuring points, since the block sub-matrix only contains one measuring point, its structural response information can only be reflected by itself; For a block submatrix containing multiple measurement points, the measurement points included in each block sub-matrix are recorded as cross-correlated measurement points. The structural responses between them

all have strong correlation, and their correlations are all greater than the set correlation threshold. An optimal measurement point can be selected in the block sub-matrix to reflect the structural response information of other measuring points in the same sub-matrix. In this way, measurement point optimization can be carried out according to the following principles:

(1) For the first and second types of measuring points, select all of them as points to be measured.

(2) For the third and fourth types of measuring points, respectively calculate the sum of the correlation coefficient values of the rows in the clustering correlation matrix where the structural responses of the corresponding measuring points in each block sub-matrix are located. If the sum of the correlation coefficient values in the row is the largest, it means that the measuring point contains more monitoring information. Therefore, the structural response position with the maximum value is selected as the measurement point to be deployed.

(3) Since the optimal measuring points among the second type of measuring points and the third type of measuring points can already reflect the information of the fifth type of measuring points, the fifth type measuring points are no longer selected as the measuring points to be deployed.

Fig. 3 is optimized according to the above principles. The initial 10 measuring points can be optimized to A1, A4, A5, and A8, with only 4 measuring points.

4. Optimization of construction monitoring measurement points

4.1. Initial plan for measuring points of construction monitoring

In most cases, the stress measurement points during the construction process of long-span spatial structures are mainly arranged in areas based on the structural installation plan, and based on the construction simulation calculation results and structural characteristics, typical representative rod arrangement measurement points are selected. The main principles to be considered are: ① Rods with high stress during construction; ② Rods with large stress changes

during construction; ③ Lift the rods near the lifting points; ④ During the partition installation process, the stress properties of the rods in the connection area may change with the construction process; ⑤ The upper chord inclined support rod at the peripheral corner end of the lifting unit; ⑥ Partially symmetrical verification test points; ⑦ Rods at other important locations.

According to the above principles, the central roof of Mall of China needs to set up a total of 104 measuring points, including 36 in the first lifting area, 22 in the second lifting area, 36 in the third lifting area, and 10 in the hoisting area, as shown in Fig.4. For the convenience of description, the measuring point number is in the form of Wa-b, which is expressed as the bth measuring point in the W-type rod of area a. The rod category W is divided into five types, with U representing the upper chord rod, D representing the lower chord rod, XF representing the oblique web rod, SF representing the vertical web rod, and XZC representing the upper chord inclined support rod.

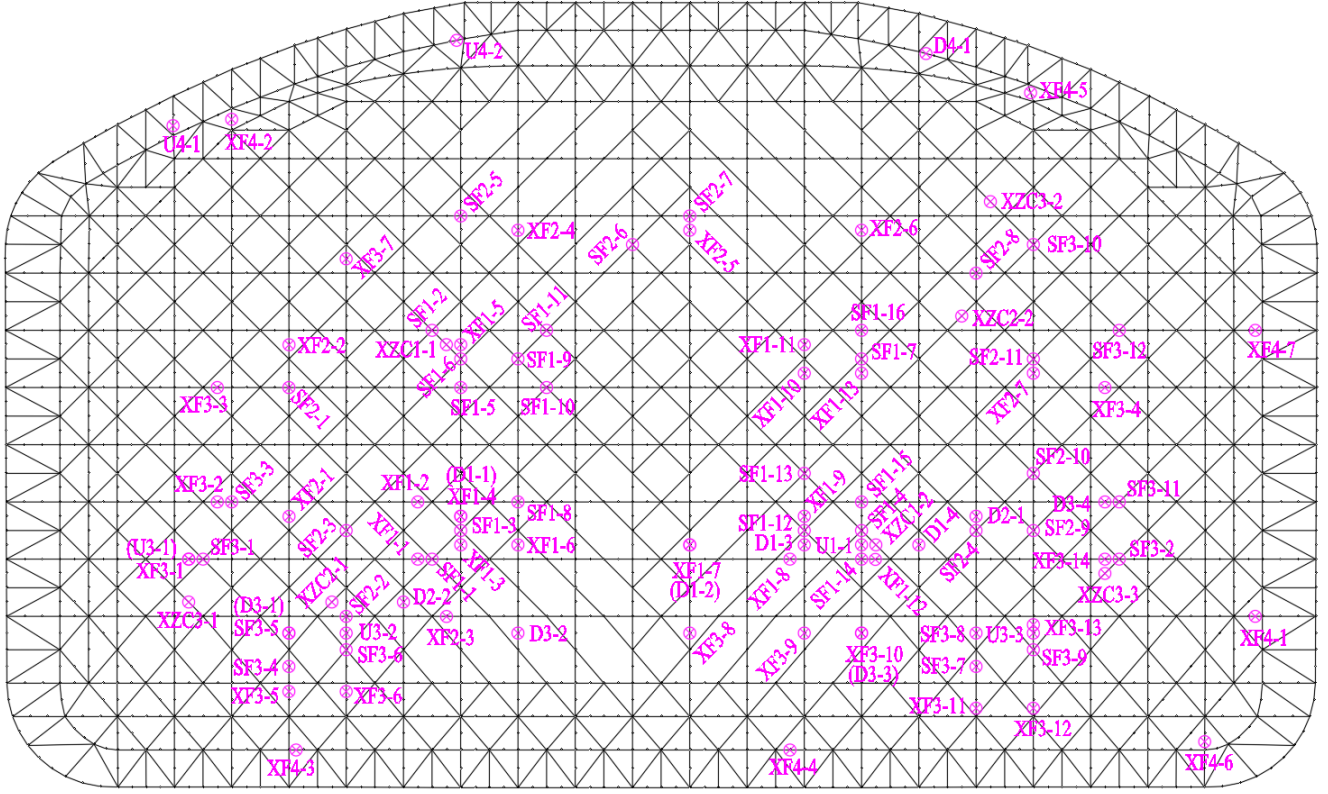


Fig. 4 Stress measuring point layout diagram

4.2. Optimization plan of measurement points

Obviously, the large number of measuring points shown in Fig. 4 brings greater difficulties to construction monitoring. Therefore, 104 measurement points were optimized based on the correlation between structural responses according to the method in Section 2.

According to the construction plan shown in Fig.1, the structural construction process is divided into four stages. Considering that each partition has a sequence of installation, the number of construction stages experienced by each partition is different. Therefore, the correlation analysis and optimal arrangement of the measuring points in each partition are carried out. To this end, the finite element software ANSYS is first used to track and simulate the entire process of structural construction, and structural response data is obtained. Referring to the correlation thresholds used in engineering cases from Reference [13] that are highly similar to the present study in terms of structure and technology, binary correlation matrices were generated by setting $\lambda = \mu = 0.6$, 0.7, and 0.8, respectively. Based on the method described in Section 2, clustering correlation matrices with block-wise characteristics were calculated for the measurement points in each partition across multiple construction stages under different threshold values. Subsequently, following the classification approach outlined in Section 2.2 for mutually correlated measurement points, the final optimized measurement point configurations corresponding to different thresholds were obtained. The results show that when $\lambda = \mu = 0.6$, the number of measurement points was reduced by 35; when $\lambda = \mu = 0.7$, the reduction was 26; and when $\lambda = \mu = 0.8$, the reduction was 16.

From the perspective of the number of reduced measurement points alone, a lower threshold corresponds to a greater reduction in the number of

measurement points. To further select the optimal threshold, the Normalized Root Mean Square Error (NRMSE) was used for comparison. Taking the measured stress data from all 104 measurement points as the complete dataset, the clustering correlation matrix was divided into submatrices based on their distribution, with each submatrix representing a distinct clustering type. From each cluster, the measurement point with the highest sum of correlation coefficients in its row was selected as the representative point. The collection of all representative points formed the simplified measurement point set, where the number of points equaled the number of clusters. The measured data corresponding to these representative points constituted the final simplified dataset. Finally, the Root Mean Square Error (RMSE) between the complete 104 - point dataset and the simplified measurement point set was calculated using Equation (22), followed by the computation of NRMSE as described in Equation (23). The resulting errors for different thresholds are presented in Table 1.

$$RMSE = \sqrt{\frac{1}{n} \sum_{t=1}^n (x_t - \hat{x}_t)^2} \quad (22)$$

In the above formula:

n : the number of data samples;

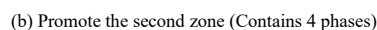
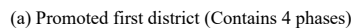
x_t : the true value (measured stress data from the complete dataset);

$\hat{x}_t$: the predicted value (representative value from the simplified measurement point set, e.g., data from representative points selected via clustering).

choice.

Table 1
Optimization Results Under Different Thresholds

Fig.5 shows the clustering correlation matrix obtained by cluster analysis using BEA after binarization when $\lambda = \mu = 0.7$. Then, the mutually - correlated measurement points are classified according to Section 2.2, as shown in Table 2.



U3-1	1	0.33	0.67	0	0	0	0.33	0.33	0.33	0.33	0.33	0	0.33	0	0	0.33	0.33	0.33	0	0	0.33	0.67	0	0	0	0.33	0.67	0.33	0.33	0	0	0.33	0	0	0.33	
U3-2	0.33	1	0.67	0.33	0	0	0	0	0	0	0.33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.33	0	0	0	0	0	0.33		
U3-3	0.67	0.67	1	0	0	0	0	0	0	0.67	0.67	0	0.33	0	0	0.33	0.33	0.33	0	0	0	0.33	0	0	0	0	0	0.33	0.67	0	0	0	0.33	0	0.33	
D3-1	0	0.33	0	0	1	0	0.33	0.33	0	0	0	0.33	0	0.33	0.33	0	0	0.33	0	0	0	0	0	0	0	0	0	0	0.33	0	0	0.33	0	0		
D3-2	0	0	0	0	1	1	0	0.33	0	0	0	0	0.33	0	0	0.33	0.33	0.67	0	0.67	0	0	0	0	0	0	0	0	0.67	0.67	0	0	0.33	0		
D3-3	0	0	0	0.33	1	1	0	0.33	0	0	0	0	0.33	0.67	0	0	0.67	0	0.67	0	0	0	0	0	0	0	0	0	0	0.67	0.67	0	0	0.67	0	
D3-4	0.33	0	0	0.33	0.33	0.33	1	0	0	0	0	0	0	0.33	0	0	0.67	0	0.33	0.33	0	0.33	0	0	0	0	0	0	0	0.33	0.33	0.33	0	0.33	0.33	
SF3-1	0.33	0	0	0	0	0	0	1	1	1	0	0	0	0	0.33	0.67	0.33	0	0.33	0.33	0.33	0.33	0	0.33	0.33	0.33	0.67	0	0	0.67	0	0	0	0	0	
SF3-2	0.33	0	0	0	0	0	0	1	1	0.67	0	0.33	0	0	0.33	0.67	0.67	0	0.67	0	0.33	0.33	0.67	0	0	0.33	0.33	0.33	0	0	0.67	0.33	0	0	0	
SF3-11	0.33	0	0.67	0	0	0	0	1	0.67	1	1	0	0.33	0	0	0.33	0.67	0.67	0	0.67	0.33	0.33	0	0.67	0	0.33	0.33	0.33	0.33	0	0.33	0.33	0	0	0	
SF3-12	0.33	0.33	0.67	0	0	0	0	0	0	1	1	0	0	0	0	0.33	0.67	0	0	0.33	0	0	0.67	0	0	0	0	0	0	0	0	0	0.33	0	0.33	
SF3-3	0	0	0	0.33	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0.33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
SF3-4	0.33	0	0.33	0	0	0	0	0.33	0.33	0	0	1	0.33	0.33	0.67	0.33	0.33	0	0	0	0.33	0	0.33	0.33	0.33	0.33	0.33	0	0	0	0.33	0	0	0	0	
SF3-5	0	0	0	0.33	0.33	0.33	0	0	0	0	0	0.33	1	1	0	0	0	0.33	0	0	0	0	0	0	0	0	0	0	0.33	0.33	0	0.33	0	0	0	
XF3-6	0	0	0	0.33	0	0.67	0.33	0	0	0	0	0.33	1	1	0.33	0	0	0.33	0	0.33	0	0	0.33	0	0	0	0	0	0	0.67	0.67	0	0	0.33	0	
SF3-6	0.33	0.33	0.33	0	0	0	0.33	0.33	0.33	0.33	0.33	0.67	0	0.33	1	0.67	0.67	0	0	0	0.33	0	0.33	0.33	0.33	0	0.33	0.67	0.33	0	0	0.33	0.33	0	0	0
SF3-7	0.33	0	0.33	0	0.33	0	0	0.67	0.67	0.67	0.67	0	0.33	0	0	0.67	1	1	0	0.67	0	0.33	0	0.33	0	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0	0	
SF3-9	0.33	0	0.33	0	0.33	0	0	0.33	0.67	0.67	0	0	0.33	0	0.67	1	1	0	0.33	0	0.33	0	0.67	0	0	0.33	0.67	0.33	0.33	0	0.67	0.67	0.67	0	0	
SF3-8	0	0	0	0.33	0.67	0.67	0.67	0	0	0	0	0	0.33	0.33	0	0	1	0	0.67	0	0	0.33	0	0	0	0	0	0	0.67	0.67	0	0.33	0	0	0	
SF3-10	0	0	0	0	0	0	0	0.33	0.67	0.67	0.33	0	0	0	0	0.67	0.33	0	1	0	0	0.67	0.33	0.33	0	0.33	0	0.67	0.33	0	0.33	0.33	0	0	0	
XF3-1	0.33	0	0	0	0.67	0.67	0.33	0.33	0	0.33	0	0.33	0	0	0.33	0	0.67	0	1	0	0	0	0	0	0	0	0	0	0.67	0.67	0	0.33	0	0	0	
XF3-2	0.67	0	0.33	0	0	0	0.33	0.33	0.33	0.33	0	0.33	0	0	0.33	0.33	0.33	0	0	1	0.33	0.33	0	0	0.33	0.33	0	0	0.67	0.33	0	0.33	0.33	0	0.33	
XF3-3	0	0	0	0	0	0	0	0.33	0.33	0	0	0	0	0	0	0	0	0.67	0	0.33	1	0.33	0	0	0.33	0.33	0	0	0	0.33	0.33	0	0	0	0	
XF3-4	0	0	0	0	0	0	0.33	0.67	0.67	0.67	0.67	0	0.33	0	0	0.33	0.33	0.67	0	0.33	0	0.33	0.33	1	0	0	0.33	0.33	0	0	0.67	0.67	0	0	0	0
XF3-5	0	0	0	0	0	0	0.33	0	0	0	0	0	0.33	0.33	0.67	0.33	0	0.33	0	0	0	0	0	1	0	0	0	0	0.33	0.67	0	0	0	0	0	
XF3-7	0.33	0	0	0	0	0	0.33	0	0.33	0	0.33	0	0.33	0	0	0.33	0	0	0	0	0	0	0	0	0	1	0.33	0.33	0.33	0	0	0	0	0	0	
XF3-8	0.67	0.33	0.33	0	0	0	0.33	0.33	0.33	0	0.33	0	0	0.33	0.33	0.33	0	0	0.33	0.33	0.33	0.33	0.33	1	0.33	0.33	0	0.33	0	0.33	0	0	0	0	0	
XF3-9	0.33	0	0.67	0	0	0	0.33	0.33	0.33	0.33	0	0.33	0	0	0.67	0.33	0.67	0	0.67	0	0.33	0.33	0.33	0	0.33	0.33	1	1	0	0	0.67	0	0	0	0	
XF3-10	0.33	0	0	0	0	0	0.67	0.33	0.33	0	0	0	0	0.33	0.33	0	0.33	0	0	0	0	0	0	0	0.33	0.33	1	1	0	0	0.67	0.33	0	0	0	
XF3-11	0	0	0	0.33	0.67	0.67	0.33	0	0	0	0	0	0.33	0.67	0	0.33	0.33	0.67	0	0.67	0	0	0	0.33	0	0	0	0	0	0	0	1	1	0	0.33	0
XF3-12	0	0	0	0	0	0.67	0.33	0	0	0	0	0	0.33	0.67	0	0.33	0	0.67	0	0.67	0	0	0	0	0.67	0	0	0	0	0	0	0	1	1	0	0
XF3-13	0.33	0	0	0	0	0	0.67	0.67	0.33	0	0.33	0	0	0.33	0.33	0.67	0	0.33	0	0.67	0.33	0.67	0	0.33	0.67	0.67	0	0	1	0.67	0	0	0.33	0	0.33	
XF3-14	0	0	0	0	0	0	0.33	0	0.33	0.33	0	0	0	0	0.33	0.33	0.67	0	0.33	0	0.33	0.33	0.67	0	0.33	0	0	0.33	0	0	0.67	1	0	0	0.33	
XZC3-1	0	0	0.33	0.33	0	0.67	0.33	0	0	0	0.33	0	0.33	0.33	0	0	0.33	0	0.33	0	0	0.33	0	0	0	0	0	0	0.33	0	0	0	1	0	0	0.33
XZC3-2	0	0	0	0	0.33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.33	0	0	0	0	0	0	0	0	0	0	0	0	1	0.67	
XZC3-3	0.33	0	0.33	0	0	0.33	0	0	0	0	0.33	0	0	0	0	0	0	0	0	0	0	0.33	0	0	0	0	0	0	0	0	0	0.33	0.33	0	0.67	1

(c) Promote the third zone (Contains 3 phases)

U4-2	1	1	1	1	0.5	0.5	1	0.5	0	0.5
XF4-4	1	1	1	1	0.5	0.5	0.5	0.5	0.5	0.5
XF4-7	1	1	1	1	0.5	0.5	0.5	0.5	0	0.5
U4-1	1	1	1	1	0.5	0.5	0.5	0.5	0.5	0.5
XF4-2	0.5	0.5	0.5	1	1	1	0.5	0.5	0.5	0.5
XF4-5	0.5	0.5	0.5	1	1	1	0.5	0.5	0.5	0.5
D4-1	1	0.5	0.5	0.5	0.5	0.5	1	1	0	0.5
XF4-1	0.5	0.5	0.5	0.5	0.5	0.5	1	1	0	0.5
XF4-3	0	0.5	0	0.5	0.5	0.5	0	0	1	0
XF4-6	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0	1

(d) Bulk area (Contains 2 phases)

Fig. 5 Cluster correlation matrix of zone measuring points

Table 2

Test point classification results table

Station category	Station number
Test points of the first kind	U1-1, D1-2, D1-3, SF1-2, SF1-3, SF1-5, SF1-6, SF1-12, SF1-15, SF1-16, XF1-2, XF1-3, XF1-4, XF1-7, XF1-8, XF1-11, XF1-13, XZC1-2, D2-1, D2-2, SF2-2, SF2-3, SF2-4, SF2-5, SF2-6, SF2-7, SF2-8, SF2-9, SF2-10, SF2-11, XF2-1, XF2-2, XF2-5, XF2-6, XF2-7, XZC2-1, XZC2-2, U3-1, U3-2, U3-3, D3-1, D3-4, SF3-3, SF3-4, SF3-6, SF3-8, SF3-10, XF3-1, XF3-2, XF3-3, XF3-4, XF3-5, XF3-7, XF3-8, XF3-13, XF3-14, XZC3-1, XZC3-2, XZC3-3, XF4-3, XF4-6
The second type of measuring point	XF1-1, XF1-5, XF1-9, XF2-4, U4-1
The third type of measuring point	{SF1-1, SF1-7}
The fourth type of measuring point	{D1-1, SF1-8}, {SF1-4, SF1-14}, {SF1-10, SF1-11}, {D3-2, D3-3}, {SF3-1, SF3-2}, {SF3-5, XF3-6}, {SF3-7, SF3-9}, {SF3-11, SF3-12}, {XF3-9, XF3-10}, {XF3-11, XF3-12}, {D4-1, XF4-1}
The fifth test point	D1-4, SF1-9, SF1-13, XF1-6, XF1-10, XF1-12, XZC1-1, SF2-1, XF2-3, U4-2, XF4-2, XF4-4, XF4-5, XF4-7

Combining Fig.4 and Fig.5, it can be found that the cross-correlated measuring points in each block sub-matrix are close to symmetrical or adjacent relationship in spatial position. It shows that there is indeed a large correlation between the structural response information of certain spatial symmetry positions and adjacent positions, which illustrates the effectiveness of the correlation analysis method to a certain extent.

According to the aforementioned layout principle of optimal measuring points, except that the first and second types of measuring points are directly determined as the optimal measuring points. One measuring point in each block sub-matrix of the third and fourth types will be selected as the optimal measuring point. After calculating and comparing the structural response of the corresponding measuring points in each block submatrix and the sum of the correlation coefficients of the rows in the clustering correlation matrix, D1-1,

SF1-4, SF1-7, SF1-11, D3-3, SF3-2, SF3-7, SF3-11, XF3-6, XF3-9, XF3-11 and D4-1 are the optimal measuring points. In addition to the auto-correlated measuring points, the measuring points selected from the cross-correlated measuring points and their corresponding related measuring points are shown in Table 3. Furthermore, the number of measurement points after final optimization was reduced to 78, as shown in Fig.6. Compared with the initial plan, it was reduced by 25%.

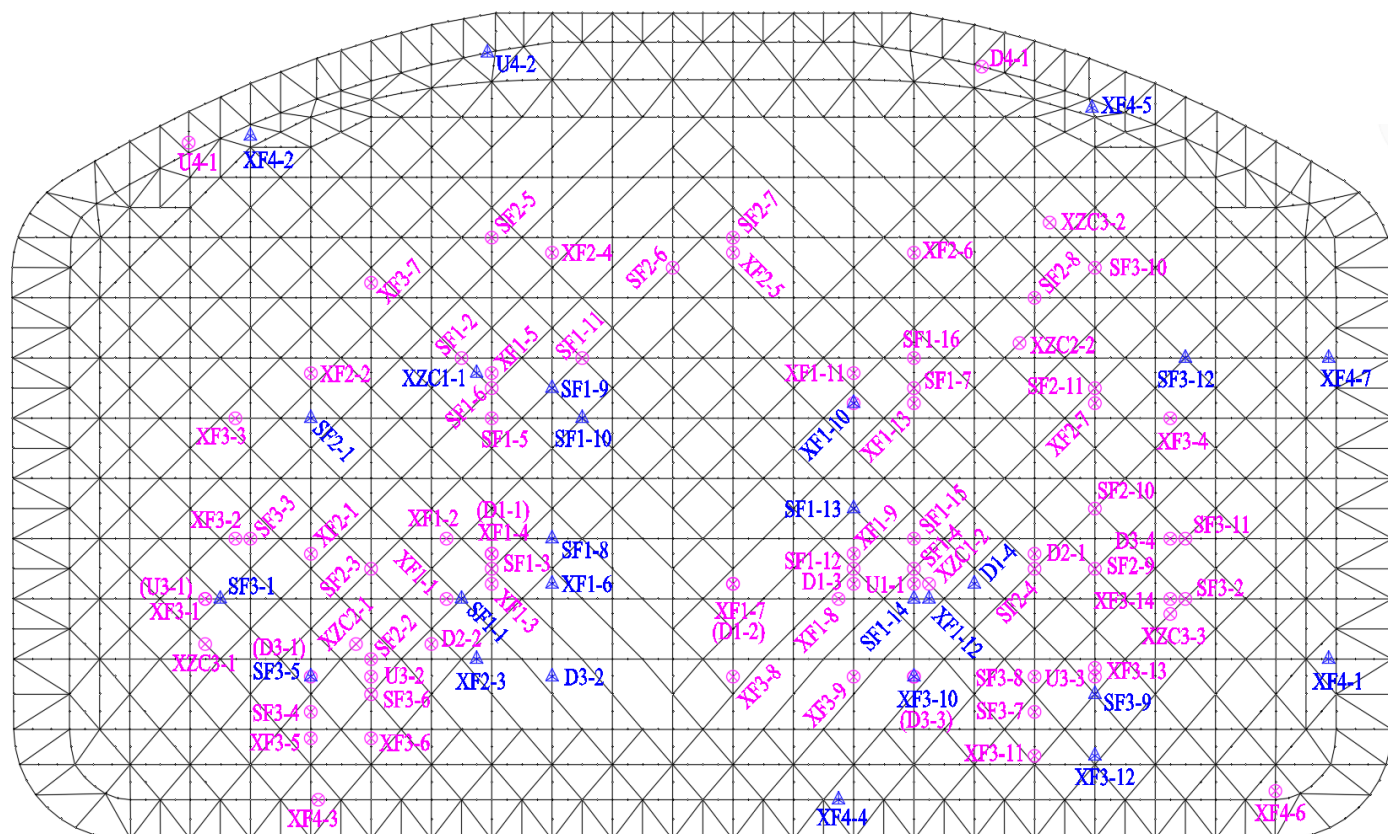
Although the measurement point reduction ratio in this study is lower than those reported in Ref.[1–4], a 25% reduction still achieves satisfactory results for the central roof steel structure of Mall of China, which features complex spatial geometry, is subjected to diverse loading conditions, and requires a comprehensive evaluation of correlations among all structural points; unlike previous studies in the literature that primarily focus on symmetric structures, single-type loading scenarios, or the optimization of localized components, this research targets the entire roof system — a structure characterized by relatively weak mechanical correlations among its components and greater challenges in optimization, and in practical applications, the extent of measurement point reduction should be determined through a trade-off between monitoring accuracy, data integrity, and implementation cost, rather than pursuing an excessively high reduction ratio in isolation; by integrating binary correlation matrix processing with BEA, an approach validated in Ref.[2] and [12], the proposed method achieves efficient measurement point reduction while maintaining monitoring effectiveness, thereby demonstrating its suitability for structural health monitoring in complex structural systems.

It is noteworthy that both the current study and the research in Ref. [13] focus on small-to-medium spatial structures, in which BEA and clustering algorithms demonstrate efficient optimization of modal measurement points. For super-large structures, however, the original BEA algorithm faces dual bottlenecks in computational complexity and dense matrix memory consumption, which may compromise feasibility due to excessive runtime or memory overflow. The scalability of the algorithm can be improved through the use of sparse matrices, parallel computing, and divide-and-conquer strategies. While the proposed method has been validated for medium-scale structures, its applicability to super-large structures relies on these optimization techniques as a direction for future research.

Table 3

Measuring point layout points and corresponding relevant measuring point information table

Test point layout point	Corresponding correlation measuring points	Test point layout point	Corresponding correlation measuring points
D1-1	SF1-8	SF3-2	SF3-1
SF1-4	SF1-14	SF3-7	SF3-9
SF1-7	SF1-1, SF1-9, XF1-9	SF3-11	SF3-12
SF1-11	SF1-10	XF3-6	SF3-5
XF1-1	D1-4, SF1-13, XF1-6	XF3-9	XF3-10
XF1-5	XZC1-1, XF1-12	XF3-11	XF3-12
XF1-9	SF1-1, SF1-7, XF1-10	U4-1	U4-2, XF4-4, XF4-7, XF4-2, XF4-5
XF2-4	SF2-1, XF2-3	D4-1	XF4-1
D3-3	D3-2		



Note: “” in the figure represents the test points determined after optimization
“” indicates the number of points that can be reduced after optimization

Fig. 6 Schematic diagram of optimal arrangement of measuring points

5. Response prediction based on LSTM neural network

In order to verify the reliability of the aforementioned optimization plan of measurement points, the response correlation between cross-correlated measurement points needs to be verified. The LSTM neural network technology, which is good at solving timing problems in deep learning, is used to establish a response prediction model. The on-site monitoring response values of the optimized measuring points are used to predict the response values of the measuring points that are eliminated during the optimization process but are cross-correlated with the optimized measuring points. At the same time, in addition to arranging sensors at optimized measuring points during the construction process, this paper also deliberately arranged sensors at cross-correlated measuring points, and compared the on-site monitoring data of these measuring point with its corresponding predicted values to verify the feasibility of optimization method of measuring points.

5.1. On-site monitoring

Since the bar of the space grid structure is mainly subjected to axial force, but the strain sensor is arranged in the actual construction. To facilitate intuitive interpretation, the measured strain values are converted into stress values in this

paper. The strain sensor arranged in this paper has a measuring accuracy of 0.1%F. S and a resolution of 0.025%F. S, as shown in Fig.7. It is arranged in the middle position along the axis of the rod of its measuring point. The entire construction process is based on time series, with a total of 250 sets of data collected in the first construction stage, 500 sets of data in the second stage, 300 sets of data in the third stage, and 300 sets of data in the fourth stage. Due to the large amount of data, this paper does not list the detailed monitoring data due to space limitations.

**Fig. 7** The strain sensor

5.2. LSTM neural network structure

Long short-term memory (LSTM) neural network is a special recurrent neural network suitable for processing and predicting time series problems[14–15]. As illustrated in Fig.8, the architecture consists of three control gates—the forget gate, input gate, and output gate—whose activation functions follow the classical LSTM design paradigm[16–18]. Specifically, the gating units employ the Sigmoid function (output range: 0 to 1), while the state units utilize the Tanh function (output range: -1 to 1). This configuration combines mathematical rigor with engineering advantages: the saturating nature of the Sigmoid function inherently aligns with gating logic, where 0 denotes complete information blocking and 1 indicates full transmission, while the zero-centered output and stronger gradient (maximum 1.0) of the Tanh function facilitate state updates and mitigate gradient vanishing in long-term dependency learning. Through these gating mechanisms, the LSTM achieves selective information transmission and state updating.

The forget gate regulates information flow from the previous cell state C_{t-1} to the current cell state C_t , with its output F_t calculated by the sigmoid function as shown in Formula (24) to represent the retention probability within the range of 0 to 1. The input gate controls the integration of new input X_t into the cell state C_t through a two-layer structure: the sigmoid layer (Formula (25)) generates an update factor I_t to realize weighted control over new inputs; the tanh layer generates candidate memory $\hat{C}_t$ (expressed by Formula (26)), whose output range of -1 to 1 can capture positive and negative features, and its stronger gradient property helps alleviate the gradient vanishing problem. The output gate regulates information output, with its output O_t calculated by the sigmoid function (Formula (27)), while the final hidden state H_t is obtained by scaling the cell state via the tanh function as shown in Formula (28), ensuring output range constraint and feature consistency.

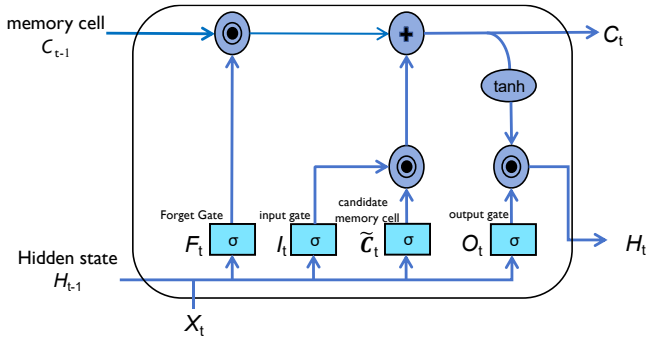


Fig. 8 Structural schematic diagram of LSTM

Note: σ in the figure represents the sigmoid function, and $\tanh$ represents the hyperbolic tangent function.

$$F_t = \sigma(W_f \cdot [H_{t-1}, X_t] + b_f) \quad (24)$$

$$I_t = \sigma(W_i \cdot [H_{t-1}, X_t] + b_i) \quad (25)$$

$$\hat{C}_t = \tanh(W_c \cdot [H_{t-1}, X_t] + b_c) \quad (26)$$

$$O_t = \sigma(W_o \cdot [H_{t-1}, X_t] + b_o) \quad (27)$$

in the above formula, W represents the network weight matrix, and b represents the network bias vector.

$$H_t = O_t \cdot \tanh(C_t) \quad (28)$$

The cell state C_t is continuously updated from the C_{t-1} of the previous moment according to the above process, and its update process is shown in formula (29).

$$C_t = F_t \cdot C_{t-1} + I_t \cdot C_t \quad (29)$$

When using the LSTM neural network to establish a correlated prediction model between measurement points for the central roof steel structure of Mall of China, the input data should be prepared first. In order to ensure the training effect, the input data should be sufficient. The principle of prediction is mainly

to selectively memorize or forget training data, write down important information, and forget information of little value. By learning important information, long-term memory can be formed, which can then play a predictive role[19].

The actually arranged measuring points on the structure and their corresponding cross-correlated measuring points have a strong correlation in each construction stage. However, considering that the correlation rules in different construction stages may change significantly, each construction stage is analyzed separately. For illustrative purposes, the prediction process of the LSTM neural network is demonstrated using the cross-correlation monitoring points D1-1/SF1-8 and U4-1/XF4-4 as examples. Here, D1-1/SF1-8 belongs to the promoted zones I and II, encompassing four construction stages with 250, 500, 300, and 300 data samples collected in stages 1 to 4, respectively; U4-1/XF4-4 is located in the bulk material zone, comprising two construction stages with 500 and 350 data samples in stages 1 and 2, respectively. During training, the monitoring response data from points D1-1 and U4-1 are used as model inputs, while the corresponding response data from points SF1-8 and XF4-4 serve as outputs, thereby establishing a response prediction model between the respective monitoring points. The monitoring data at each stage are used as model data sets for training. For each stage, 1/5 of the data is used as the test set. The selection principle of test set is $t=5d$ ($d=1, 2, 3, \dots$), that is, every fifth group of data is taken as the test set, and the remaining data as the training set.

It is worth mentioning that regularization can make the gradient of the optimization loss function more stable to a certain extent, thereby improving the training speed and accuracy of the model[20], at the same time, regularization can also make the data in the model tend to be unified in the end, which is convenient for evaluation. Therefore, the maximum and minimum regularization method is used to standardize all data during the training process, and the processing process is shown in formula (30).

$$x' = \frac{x - x_{\min}}{x_{\max} - x_{\min}} \quad (30)$$

In the formula, x is the monitoring response data of the measuring point; $x_{\min}$ is the minimum value in the monitoring response data group of the measuring point; $x_{\max}$ is the maximum value in the monitoring response data group of the measuring point.

In addition, during the training process of LSTM model, parameters such as the number of network layers, the number of hidden layer units, the batch size, the maximum number of iterations, the learning rate, the loss function and the optimizer in the model all directly affect its training effect, so there is a necessity for set and optimization. In particular, the choice of loss function and optimizer is crucial to the normal operation of the model, because the loss function is used to measure the difference between the model's predicted output value and the true value. The essence of the LSTM training process is to minimize the loss function. At present, common loss functions include mean square error function (MSE) and cross entropy function (CE). MSE is often used in regression tasks, and CE is often used in classification tasks. This paper chooses the loss function of MSE, as shown in formula (31). In terms of selecting an optimizer, the gradient descent algorithm of adaptive moment estimation (Adam) is currently the most commonly used. This algorithm has the advantages of fast convergence speed, small memory usage, and better learning effect. Therefore, this paper chooses the Adam algorithm to optimize the training of the model.

$$\text{MSE} = \frac{1}{n} \sum_{t=1}^n (x_t - \hat{x}_t)^2 \quad (31)$$

In the formula, n is the number of samples; x_t is the actual monitoring value; and $\hat{x}_t$ is the predicted model value.

In terms of model performance evaluation, the root means square error (RMSE) and the mean absolute error (MAE) can usually be selected to evaluate the model prediction accuracy, and their calculations are shown in formulas (32) and (33) respectively. Among them, RMSE can reflect the degree of dispersion between the predicted model value and the real value; MAE can reflect the closeness between the predicted model value and the real value. The smaller the value of each indicator is, the smaller the error between the predicted model value and the real value is, and the better the training effect is.

$$\text{RMSE} = \sqrt{\frac{1}{n} \sum_{t=1}^n (x_t - \hat{x}_t)^2} \quad (32)$$

$$\text{MAE} = \frac{1}{n} \sum_{t=1}^n |x_t - \hat{x}_t| \quad (33)$$

During the specific training process, the LSTM neural network prediction

model was constructed using Matlab software (its architecture shown in Fig.9), which takes input features and processes them sequentially through an LSTM layer to capture temporal dependencies, followed by a Dropout layer to randomly discard neurons and suppress overfitting, then another LSTM layer to further refine feature learning, another Dropout layer to further enhance generalization, and finally a fully connected layer to integrate features and generate output for effective sequence data processing and feature extraction.

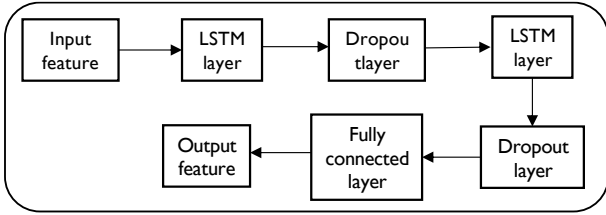


Fig. 9 Structural of the LSTM prediction model

The grid search method[21] was employed to iteratively optimize the number of network layers, hidden layer units, maximum iterations, and learning rate, with key parameters selected as shown in Table 4. The optimal values were selected based on the MSE from 5-fold cross-validation, where validation subsets were exclusively partitioned from the training set. A 2-layer LSTM architecture was adopted, reducing cross-validation MSE by 12.7% compared to a single layer and enhancing complex pattern fitting. The unit configuration (15,5) yielded a stable plateau in cross-validation MSE during later training stages (fluctuation range $< \pm 0.003$), effectively balancing model accuracy with computational efficiency. Dropout layers (rate = 0.2) were applied after each LSTM layer for regularization[22], achieving minimal cross-validation MSE fluctuation (± 0.003) and optimal overfitting suppression. A learning rate of 0.001 ensured stable gradient updates and a cross-validation MSE of 0.080; higher rates (≥ 0.01) caused gradient oscillation, while lower rates (0.0001) hindered convergence. Given the moderate training sample size, full-batch training was employed. Final hyperparameter evaluation on an independent test set confirmed generalization stability, with test MSE consistent with cross-validation trends.

Model convergence exhibits three distinct phases—rapid learning, stable convergence, and saturation plateau. As shown in Fig.10, epochs 0–500 involve rapid fitting of basic temporal features, with RMSE and loss decreasing sharply; epochs 500–3000 see the model capturing core patterns, slowing RMSE decline as it enters the diminishing returns zone; and epochs 3000–4000 maintain RMSE below 0.1 with minimal fluctuation, indicating convergence saturation. Further training risks overfitting, wastes computational resources, and yields no accuracy gains, thus the optimal maximum training iterations are set to 4000.

Table 4
Hyperparameter Search Space

Types of hyperparameters	Search range	Optimal value
Number of LSTM layers	{1, 2}	2
Cell array combination	{(10), (15), (20), (10,5), (15,5)}	(15, 5)
Dropout rate	{0.1, 0.2, 0.3, 0.4}	0.2
Learning rate	{0.1, 0.01, 0.001, 0.0001}	0.001
Maximum number of iterations	{3000, 4000, 5000}	4000

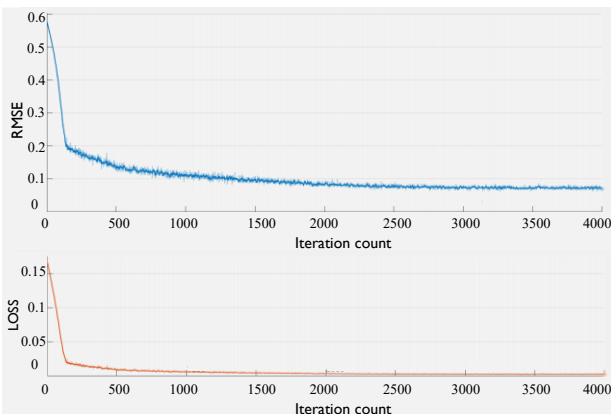


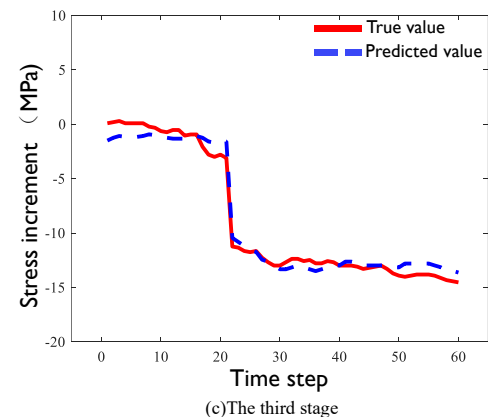
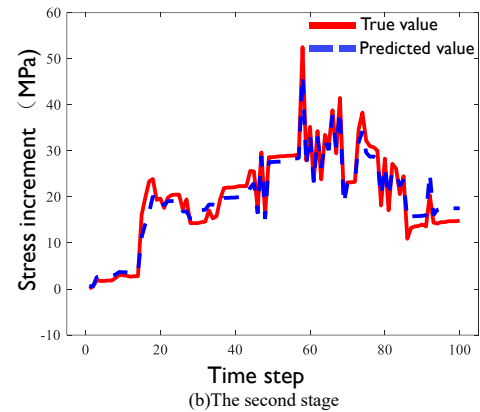
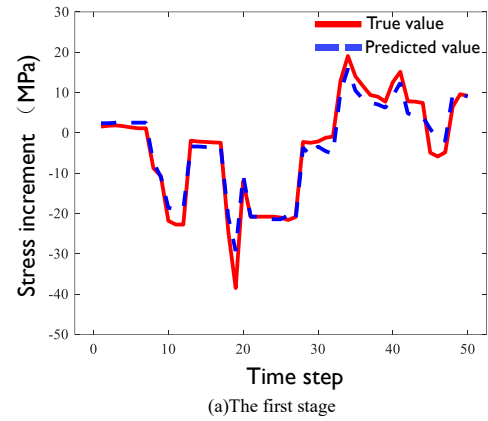
Fig. 10 The RMSE curve and loss curve during the training process

5.3. Prediction result analysis

The trained model was employed to predict the data of the test set, and the model evaluation indicators for members SF1-8 and XF4-4 at each construction stage were calculated according to Equations (32) and (33), as presented in Tables 5 and 6. The results indicate that the error metrics of the model at each stage are relatively small, demonstrating that the prediction accuracy falls within an acceptable range. To more intuitively illustrate the predictive performance of the model, a visual comparative analysis of the trends between the predicted and actual values at each stage was conducted, as shown in Fig.11 and Fig.12.

Table 5
Measurement point SF1-8 prediction error table

Construction phase	RMSE	MAE	Absolute maximum error (MPa)
Stage 1	2.7391	2.1410	9.32
Stage 2	2.2475	1.8384	6.33
Stage 3	0.8145	0.6967	1.651
Stage 4	3.2557	2.7640	6.58



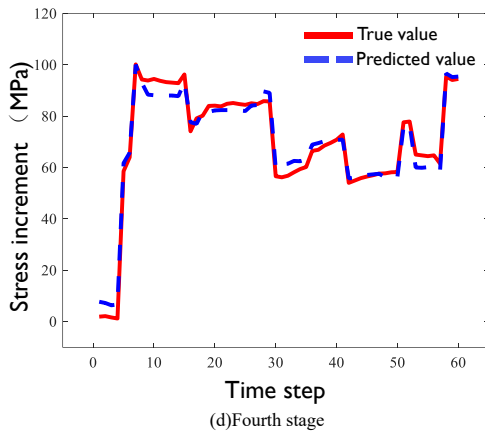


Fig. 11 SF1-8 Prediction via LSTM model

Table 6
Measurement point XF4-4 prediction error table

Construction phase	RMSE	MAE	Absolute maximum error (MPa)
Stage 1	1.4382	1.1142	3.36
Stage 2	2.2403	1.0917	2.15

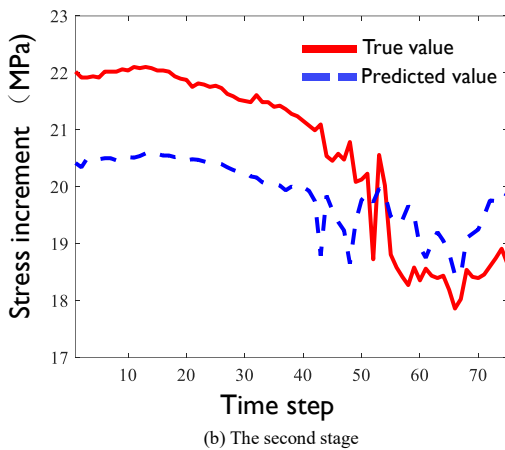
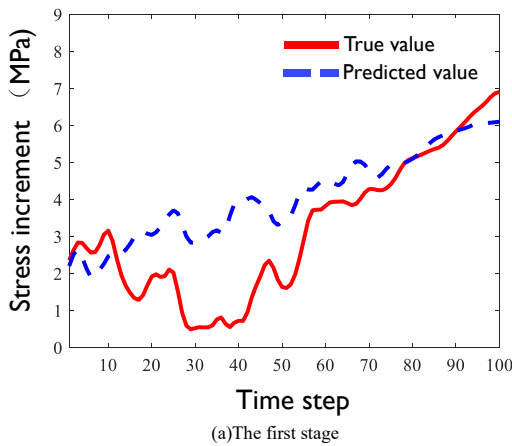


Fig. 12 XF4-4 Prediction via LSTM model

As shown in Fig.11, the Pearson correlation coefficient and Type-B grey relational degree effectively reveal the similarity in magnitude and trend of structural response variations. The model predictions closely follow the actual trends across construction stages, accurately capturing stress variations at SF1-8 despite fluctuations, thereby demonstrating the LSTM model's ability to learn the response correlation between optimized and cross-correlated measurement points. However, as shown in Fig.12, its prediction accuracy for the diagonal web member XF4-4 is lower than for SF1-8, due to XF4-4's more complex loading conditions, including combined vertical and horizontal forces, greater

sensitivity to structural deformation, and additional joint moments, leading to a more intricate stress response.

The effectiveness of the LSTM model stems from its ability to capture the dynamic evolution of response correlations among key measurement points across construction stages, reflecting the combined influence of structural system transformations, load path redistributions, and construction sequence dependencies. In Stage 1, falsework unloading in Zone II shifts the load-bearing mode from falsework support to Zone I lifting points, reducing system stiffness and weakening measurement point correlations. Stage 2 integrates Zone III into a spatial grid, significantly increasing stiffness and enhancing correlations, demonstrating the transition from discrete components to global coordination. The construction sequence critically affects load path continuity—sequential lifting (Zone I then Zone II) reduces Stage 1 correlations, while synchronous lifting mitigates this effect. Stage 3's support removal redirects load paths, causing a sharp correlation drop, while the "central-to-peripheral" Zone III installation promotes stepwise stiffness growth. By Stage 4, the self-supporting system stabilizes, fixing load distribution and correlations.

Multi-stage analysis overcomes single-stage limitations by tracking correlation network evolution: (1) detecting transient events (e.g., Stage 3's unloading-induced correlation mutations); (2) quantifying construction sequence impacts on stiffness accumulation; and (3) enabling the LSTM model to learn time-varying behaviors via stage encoding, improving prediction generalization.

5.4. Applicability verification of prediction model under accidental load

During the construction process, the structure may bear a large concentrated load in a local area because of some unexpected situations, resulting in the sudden increase of stress of some members. In order to test whether the LSTM prediction model can accurately predict the sudden accidental action response, the SF1-8 in the upgrading process of zone 1 is taken as an example to verify it.

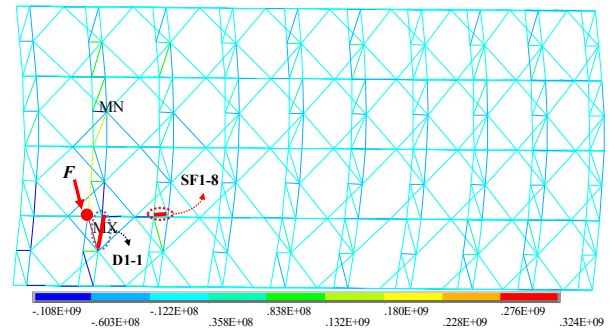


Fig. 13 Stress distribution diagram

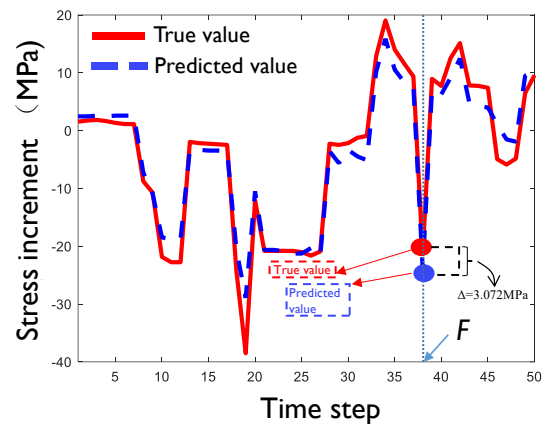


Fig. 14 Prediction of member SF1-8 during first stage

It is assumed that a vertical concentrated load $F=250\text{kN}$ is suddenly applied to the node as shown in Fig.13 during the lifting process of zone 1. The finite element calculation shows that under the action of F , the stress of D1-1 will change from -18.43MPa to -42.708MPa , and the stress increment is -24.665MPa ; while the stress of SF1-8 will change from -11.774MPa to -33.132MPa , and the increment is -21.358MPa . In actual project, a sensor is

arranged on D1-1, so we input the stress increment value of D1-1, namely, -24.665MPa into the LSTM prediction model to obtain the stress variation trend of SF1-8, as shown in Fig.14. Specifically, the predicted stress increment value is -24.43MPa at the moment F is applied. Compared with the calculated value of SF1-8, the difference is only 3.072MPa. It can be seen that LSTM also maintains a good robustness to the sudden state in the construction process.

This section focuses on validating the core capability of the proposed LSTM-based model for predicting inter-point response correlations, demonstrating its satisfactory predictive performance under a single static load condition. The model's generalizability to complex loading scenarios—characterized by dynamic time-varying properties, variable load positions, and multi-directional force coupling—remains to be verified in future research.

6. Conclusions

Based on the characteristics of certain correlation between the responses of long-span spatial structures, this paper proposes a optimization method of multi-stage construction monitoring measurement points that considers the correlation of structural responses, and uses LSTM neural network to establish a structural response prediction model. The main conclusions are as follows:

(1) Pearson correlation coefficient and B-type gray correlation can effectively explore the similarity in change size and change trend between structural responses. The structural response comprehensive correlation matrix established based on this can quantify the correlation strength of the response of measuring points at different construction stages. Moreover, it is found that the response correlation rules of relevant measuring points in different construction stages are different.

(2) The clustering correlation matrix transformed by the bond energy algorithm can quickly realize the classification and optimization of measurement points for large-span spatial structures considering multiple construction stages. Compared with the traditional layout plan of construction monitoring measurement points, the optimization method of measurement points that considers the comprehensive correlation of responses can optimize the number of measurement points by up to 25%.

(3) The LSTM neural network can well learn the response correlation between each optimized measurement point and its corresponding related measurement point, and the prediction results have good accuracy.

Funding

National Natural Science Foundation of China (51708067); 2022 Technology Innovation and Application Development Project of Nan'an District, Chongqing (2022).

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HYSTERETIC BEHAVIOR OF H-SECTION ALUMINUM ALLOY MEMBERS UNDER AXIAL LOADING: EXPERIMENTAL AND NUMERICAL STUDY

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ABSTRACT

This study investigates the hysteretic behavior of aluminum alloy members in gusset joint latticed shells. Due to the low elastic modulus of aluminum alloys and the weak-axis buckling susceptibility of H-shaped members, their hysteretic behavior is complex. However, studies remain limited, particularly on their axial compression and tension behavior under varying buckling modes. The research combines experimental and numerical approaches. Hysteretic loading tests were conducted on 11 aluminum alloy members under axial loading. A refined finite element (FE) model was developed to simulate the process, followed by a pin-ended member model for parametric analysis considering slenderness ratio, cross-sectional parameters, initial geometric imperfection, and tensile/compressive displacement amplitudes. A multi-scale latticed shell model was then constructed to evaluate the effectiveness of segmented beam elements in capturing hysteresis. Results show that compression buckling significantly reduces bearing capacity, though tensile loading mitigates residual deformation. Higher slenderness ratios and greater initial imperfections worsen post-buckling degradation and alter hysteresis curves. The multi-scale analysis confirms the accuracy of the four-segment B32OS beam element in modeling hysteresis. This study addresses a gap in hysteretic behavior research of aluminum alloy members and provides essential data and modeling guidance for the seismic design of aluminum alloy latticed shells.

ARTICLE HISTORY

Received: 30 April 2025
Revised: 17 September 2025
Accepted: 20 September 2025

KEYWORDS

Aluminum alloy members;
Cyclic loading;
Hysteretic behavior;
Finite element simulation;
Numerical Simulation;
Beam elements

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1. Introduction

Aluminum alloy gusset joint latticed shells are a novel spatial structural system known for their aesthetic appeal, corrosion resistance, and lightweight properties, increasingly used in engineering applications [1][2]. As a spatial truss structure, the members of single-layer spherical latticed shells primarily bear axial compression under static loads such as dead and live loads. However, under horizontal seismic action, latticed shells undergo cyclic dynamic loading and out-of-plane vibration, with members bearing substantial axial force and bending moment. Given that the elastic modulus of aluminum alloys is only one-third that of steel, aluminum alloy members exhibit lower buckling bearing capacity under compression-bending action compared to steel members. Additionally, aluminum alloy members are industrially extruded profiles, often with H-section shapes, where significant differences in moment of inertia between strong and weak axes make them prone to buckling about the weak axis.

Extensive research has been conducted on the static behavior of aluminum alloy members. The earliest axial compression experiments can be traced back to Osgood and Holt in the 1930s [3][4]. Today, design codes in many countries provide mature formulations for calculating the bearing capacity of axially compressed or compression-bending aluminum alloy members [5]. Over the past decade, researchers such as Guo et al. [6] performed axial compression tests on 63 profiled aluminum alloy members, creating column curves for stability calculations. Liu et al. [7][8] investigated the compressive bearing capacity and failure modes of H-type aluminum alloy members with web openings by using experimental, numerical, and theoretical methods, analyzed the effects of opening parameters, cross-section dimensions, and slenderness ratios on the bearing capacity of the members, and gave the calculation methods. Zhu et al. [9] analyzed in-plane stability considering the stiffness of aluminum alloy gusset joints and adjacent member constraints, determining the effective lengths. Sun et al. [10] studied axial compression performance under various boundary conditions for H-section aluminum alloy members with initial curvature and torsional angles, observing flexural-torsional buckling failure. Moreover, Wang et al. [11][12][13] and Adeoti [14] conducted extensive experimental studies and finite element analyses on the flexural buckling and local buckling behavior of I-section, rectangular hollow section, and circular hollow section members. They compared the analysis results with existing design standards and found that most current design codes tend to be conservative for symmetric section members. Overall, researchers have conducted in-depth and systematic studies on aluminum alloy axially compressed members, bending members, and beam-column members, particularly symmetric section members. The bearing capacity calculation methods for these members have become relatively mature, leading to the development of relevant design codes [15].

Current research on the hysteretic behavior of aluminum alloys primarily focuses on the material and joint behavior. Regarding the hysteretic behavior of

aluminum alloy materials, Guo et al. [16] conducted cyclic loading tests and fitted the monotonic loading curves, backbone curves, and hysteresis curves of 6082-T6 and 7020-T6 aluminum alloys, proposing a uniaxial hysteretic constitutive model for Chinese aluminum alloys and verifying its reliability. Pisapia [17], Branco [18], Kyriakos [19], and others investigated the low-cycle fatigue behavior and hysteretic constitutive models of various aluminum alloys through low-cyclic loading tests and theoretical analyses. For the hysteretic behavior of aluminum alloy joints, Guo [20], Wu [21], Xu [22], and Liu [23] conducted extensive experimental studies, investigating the failure modes, bearing capacity, deformation capacity, and energy dissipation capacity of gusset joints with different member cross-sections and joint configurations.

In contrast, research on the hysteretic behavior of aluminum alloy members remains scarce. Chen et al. [24] used finite element analysis to investigate the effects of alloy type and slenderness ratio on the hysteretic behavior of aluminum alloy axially compressed members. The results showed that, for the same slenderness ratio, the hysteretic behavior of 6061-T6 was superior to that of 6061-T4. Additionally, specimens with smaller slenderness ratios exhibited better performance under cyclic loading. Wu et al. [25] conducted eccentric cyclic loading tests on three H-shaped 6061-T6 aluminum alloy members with different slenderness ratios. They further used finite element (FE) model to study the effects of cross-section, slenderness ratio, and eccentricity. The results indicated that local buckling occurred first, followed by crack initiation at the buckled region, with cracks gradually propagating until the members fractured. However, due to the small slenderness ratios of the tested specimens, no global instability failure was observed. Thus, experimental research on the hysteretic behavior of the members experiencing global instability is still lacking, highlighting the need for further studies.

Several hysteretic models have been proposed for circular tubes under compression buckling and tension strengthening [26][27][28], yet these are often complex to implement and insufficiently accurate, being suitable only for single buckling modes. For H-sections prone to multiple modes, FE methods remain more adaptable. Prior works [29][30] commonly adopted beam element B31OS to represent aluminum alloy members in multi-scale models. In this study, members are subdivided into multiple beam elements to simulate buckling and hysteretic behavior, clarifying the simulation accuracy under various initial geometric imperfections and complex loading conditions.

Due to the current lack of research on the hysteretic behavior and simulation methods of aluminum alloy members during overall buckling, this study combined experimental loading tests with numerical simulations to investigate the hysteretic behavior of H-section members. First, axial loading hysteretic experiments were conducted on aluminum alloy members. Then, refined FE models were developed to simulate the loading process, with the simulation accuracy verified against the experimental data. On this basis, FE models with pin-ended member were established, and a parametric analysis was performed to investigate the effects of key parameters on the hysteretic behavior of the

members. Finally, the beam elements intended for use in the multi-scale latticed shell model were studied using both single-member and multi-scale models to determine the appropriate number of element divisions and the accuracy of numerical simulation. The novelty of this study lies in experimentally capturing global instability of aluminum alloy members and validating a multi-segment beam element model against shell simulations.

2. Experimental study

2.1. Overview of the experiment

2.1.1. Objective of the experiment

This experiment primarily focused on collecting load and displacement data to assess the stress state and geometric configuration of the members. The study examined the hysteretic loading process of aluminum alloy members, exploring the impact of slenderness ratio and loading protocols on hysteretic behavior. Additionally, the hysteresis curves obtained from the experiment were used to evaluate the accuracy of subsequent FE models.

2.1.2. Specimen design

Due to the poor weldability of aluminum alloys and significant degradation in material properties in the heat-affected zone, the weld zone tends to fracture prematurely under tension. Therefore, bolted connections were employed at the member ends. To avoid premature fracture at the bolted connection, a dog-bone-shaped specimen was designed, reducing the flange width in the middle segment with a linear transition at the variable cross-section. The schematic diagrams of the specimens are shown in Fig. 1.

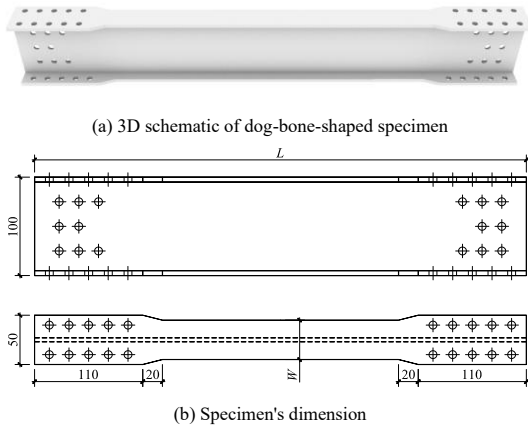


Fig. 1 Dog-bone-shaped specimen

Table 1

Specimen parameters

Specimen ID	Length L (mm)	Flange Width W (mm)	Loading Protocol
YSY-1	1000	50	Pretest - Monotonic Loading
YSY-2	1000	50	Pretest - Single-Cycle Hysteretic
YSY-3	1000	50	Pretest - Multi-Cycle Hysteretic
L1200-TestA	1200	40	Hysteretic Loading
L1200-TestB	1200	40	Hysteretic Loading
L1000-TestA	1000	40	Hysteretic Loading
L1000-TestB	1000	40	Hysteretic Loading
L700-TestA	700	35	Hysteretic Loading
L700-TestB	700	35	Hysteretic Loading
L500-TestA	500	30	Hysteretic Loading
L500-TestB	500	30	Hysteretic Loading

The cross-sectional specification of the aluminum alloy members in the loading test was H100×50×4×5, made of aluminum alloy 6061-T6. The specimen parameters are listed in Table 1. Variables included member length and loading protocols. Hysteretic loading was applied to specimens with lengths of 1200 mm, 1000 mm, 700 mm, and 500 mm, with two loading protocols for each length. The flange widths of the middle segment were 40 mm, 40 mm, 35 mm, and 30 mm, respectively. Three pretests were conducted before the formal

experiment: monotonic loading, single-cycle hysteretic loading, and multi-cycle hysteretic loading. The objective was to confirm the loading capacity of the setup, validate displacement measurement methods, and optimize the experimental procedure.

2.1.3. Material properties

The aluminum alloy members were made of 6061-T6 aluminum alloy. According to the standard "Metal Materials Tensile Test Part 1: Room Temperature Test Method" (GB/T 228.1-2021) [31], four material specimens were prepared by sampling from the same batch of aluminum alloy members, two from the flanges and two from the web.

The material test results are shown in Fig. 2 and Table 2. The statistical results are the nominal stress and strain of the material, which need to be converted into true stress and strain according to the following formula when performing numerical analysis[32].

$$\varepsilon_{\text{true}} = \ln(1 + \varepsilon_{\text{nom}}) \quad (1)$$

$$\sigma_{\text{true}} = \sigma_{\text{nom}}(1 + \varepsilon_{\text{nom}}) \quad (2)$$

Where ε_{nom} and σ_{nom} are nominal stress and strain, $\varepsilon_{\text{true}}$ and σ_{true} are true stress and strain.

The table includes E (elastic modulus), $f_{0.2}$ (proof strength at 0.2% non-proportional extension), f_u (ultimate strength), and ε_u (ultimate strain). The elastic modulus of the flanges was slightly higher than that of the web, while the ultimate strain and 0.2% proof strength were slightly lower. The difference in ultimate stress between the two was within 0.4%. Overall, the material properties of the flanges and the web were essentially consistent, allowing for the use of the same material parameters.

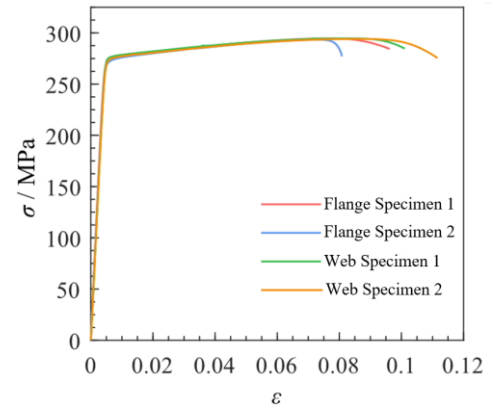


Fig. 2 Stress-strain curve of material test

Table 2

Summary of material property test results

Specimen	E / MPa	$f_{0.2}$ / MPa	f_u / MPa	ε_u
Flange Specimen 1	65629	273.8	294.6	0.096
Flange Specimen 2	63943	272.5	293.4	0.081
Web Specimen 1	61828	276.4	294.6	0.101
Web Specimen 2	62354	274.7	294.1	0.112

2.1.4. Setup of the experiment

Hysteretic tests require both tension and compression; thus, the support connections needed special design. The schematic of the experimental setup is shown in Fig. 3. The boundary conditions were configured such that the upper and lower ends were rigidly connected around the strong axis, preventing rotation and lateral displacement. The weak axis was hinged, with an ear plate and pin connection at the lower end and a single-acting hinged connection to the actuator at the upper end. Both the top and lower ends were made of Q235 steel. Two stiffening ribs were welded to the top end's connection plate to enhance its bending stiffness, and four M36 high-strength bolts were used to connect it to the actuator. Similarly, two stiffening ribs were added to the ear plates of the lower end to prevent rotation about the strong axis, and four M36 high-strength bolts were used to connect it to the test platform. The aluminum alloy members were connected to the top and lower ends using 28 M10 high-strength bolts. The

loading system consists of a 200 kN electro-hydraulic servo actuator equipped with displacement and force sensors. The displacement control accuracy was 0.01 mm, and the load control accuracy was 0.01 kN. The pretests verified that the lateral stiffness of the actuator was sufficient, making additional horizontal support unnecessary. Furthermore, there was no plastic shear deformation in the bolts, confirming sufficient connection bearing capacity.

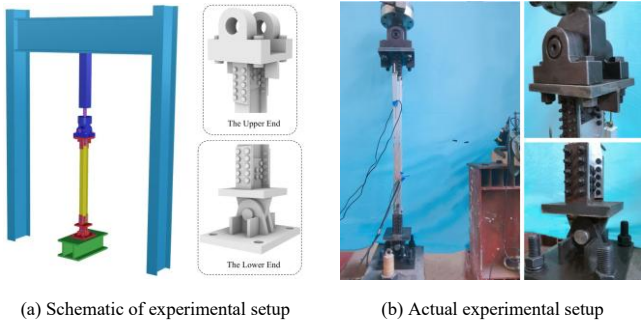


Fig. 3 Setup of the experiment

2.1.5. Loading protocol

The hysteretic loading protocol adopted a dual control approach, utilizing displacement and load criteria. During compression, displacement was the primary control target. During tension, displacement was prioritized with load as a secondary criterion. In the initial compression stage, load control was applied until approaching the peak load, then switched to displacement control until reaching the target displacement. In tension, load control was used initially, switching to displacement control near the ultimate load to prevent premature fracture due to the poor ductility of aluminum alloys.

Two loading protocols, Test A and Test B, were employed. The two loading protocols have the same amplitude as shown in Fig. 5 a. The difference is that the TestA loading system only loads once per displacement level; the TestB loading system repeats loading twice per displacement level.

2.1.6. Layout of measurement point

The primary objective of this experiment was to investigate the relationship between the vertical deformation and vertical load of the members and to plot the hysteresis curves. The vertical load was recorded using the force sensor of the actuator, which simultaneously measured the displacement at the actuator's loading end. However, the actuator displacement included numerous complex factors, such as deformation of the reaction frame, deformation of the connection supports, and various gaps. Additionally, it encompassed the vertical displacement at both ends, causing the measured actuator displacement to be significantly greater than the vertical displacement of the aluminum alloy members themselves. Therefore, displacement transducers were installed on the aluminum alloy members to measure the relative vertical displacement between the upper and lower ends.

To measure the relative vertical displacement between the upper and lower ends of the aluminum alloy members, linear variable displacement transducers were fixed to the upper end using clamps. The connection between the displacement transducers and clamps adopted a hinged installation method. This hinged configuration served two critical functions: first, it allowed the displacement transducer to move horizontally with the member when lateral displacement occurred due to buckling; second, it enabled the displacement transducer to rotate freely, maintaining a vertical measurement direction. This design ensured complete synchronization between the displacement transducer and the deformation trajectory of the member while maintaining measurement reference stability, allowing precise measurement of relative vertical displacement during buckling.

Fig. 4 shows the schematic of the displacement and strain measurement point layout. Each specimen was equipped with four displacement transducers. The displacement transducers had a measurement range of 0–50 mm and an accuracy of ± 0.01 mm, calibrated against a micrometer prior to testing. D1 and D2 were vertical displacement transducers symmetrically arranged along the central axis on the outer side of the flanges. As mentioned earlier, D1 and D2 measured the relative vertical displacement between the upper and lower ends of the aluminum alloy members. The average of the two readings was taken as the relative displacement of the members. D3 and D4 were horizontal wire displacement transducers fixed to the ground via brackets, with the wire ends attached to the web at the center of the aluminum alloy members to measure horizontal displacement at the midpoint. This data was used to determine residual deformation.

Six unidirectional strain gauges were affixed to the specimens, labeled S1

to S6, all located on the central cross-section of the members. S1 to S4 were symmetrically placed at the outer corner points of the flanges, while S5 and S6 were affixed to the web, 30 mm from the inner side of the flange. These six strain measurement points served two main purposes: first, to determine whether the specimen was loaded centrally during the preloading phase based on the measured strain data; second, to assess the tensile condition of the member during loading, preventing premature fracture.

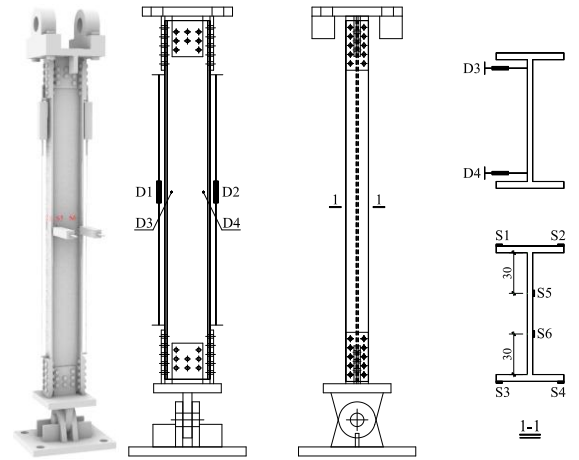


Fig. 4 Schematic of measurement point layout

2.2. Result of the experiment

Taking specimen L1000-TestA as an example, the hysteretic loading process is illustrated in Fig. 5. The loading protocol (Fig. 5(a)) was displacement-controlled by the actuator, with positive displacement applying compression and negative displacement applying tension. Each loading cycle consisted of compression loading, unloading, tension loading, and unloading. Residual deformation was observed after each cycle, with the actuator deviating from its initial position upon unloading.

To illustrate the load-displacement variation during loading, one cycle A-B-C-D-E from the hysteretic loading process is discussed. Fig. 5(b) shows the load-displacement hysteresis curve measured by the actuator, where a slip was observed near the zero-load point due to the finite stiffness of the reaction frame and connection components, as well as slip between various connectors. Fig. 5(c) presents the load-displacement curve based on the vertical relative displacement between the aluminum alloy member and the upper and lower end connections, reflecting the intrinsic hysteretic behavior of the aluminum alloy member. Fig. 5(d) displays the horizontal displacement at the midpoint of the specimen, including the member's deformation and rigid body horizontal displacement induced by end rotation.

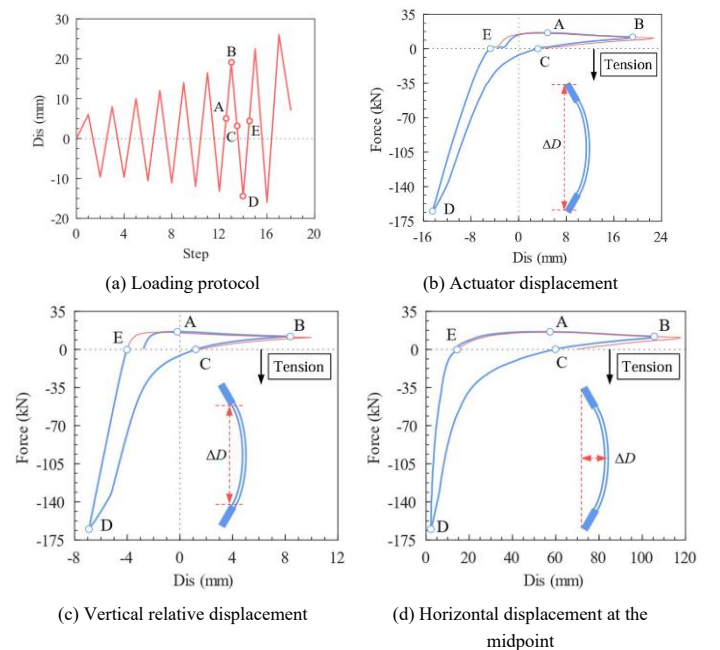
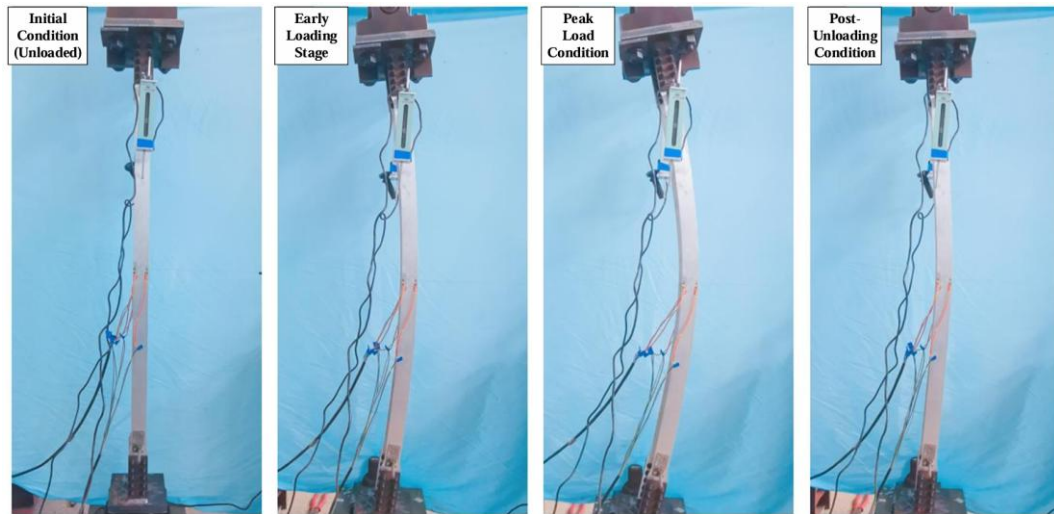


Fig. 5 Example of hysteretic loading (L1000-TestA)

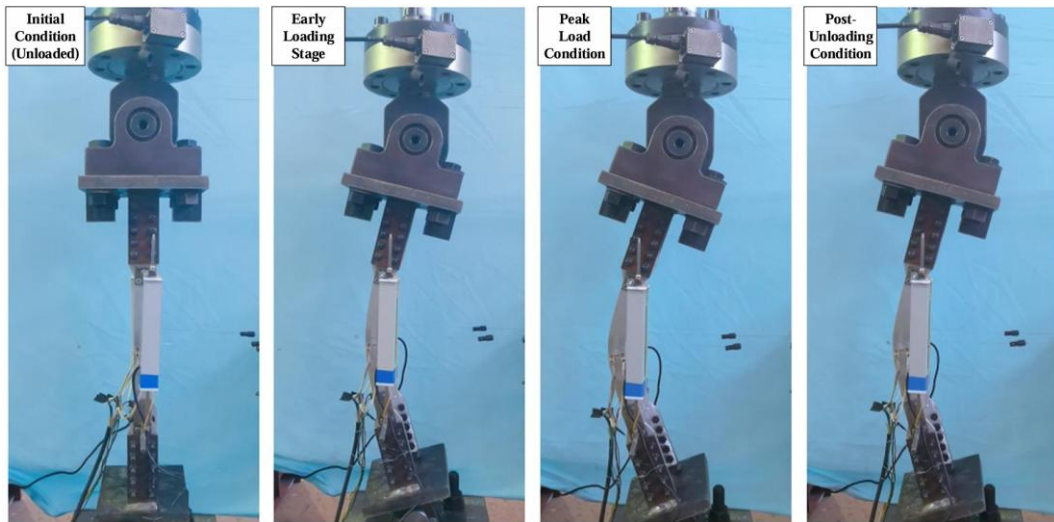
Point A represented the compression extreme. After passing point A, the member buckled, resulting in a reduction in bearing capacity along segment A-B. Segment B-C corresponded to negative displacement increase (compression unloading). At point C, the load returned to zero, yet notable residual deformation remained in both vertical and horizontal directions. In segment C-D, the specimen was subjected to tension, where plastic deformation partially alleviated residual deformation caused by compression. Segment D-E represented the tension unloading phase. Upon tension unloading to zero, elastic deformation recovery occurred, causing the member to lengthen slightly compared to the pre-compression loading phase. The horizontal deformation was essentially consistent with the pre-compression loading phase. Since

horizontal residual deformation was similar, no significant reduction in bearing capacity was observed in the subsequent hysteretic loading cycle.

Fig. 6 illustrated the deformation of specimens L1200-TestA and L500-TestB before loading and at the final loading stage. The hysteresis curves for some specimens are presented in Fig. 7, due to space constraints. The experimental data indicated that members with lower slenderness ratios exhibited higher degrees of plastic deformation under the same loading displacement, resulting in more significant residual deformation upon unloading. Members with lower slenderness ratios demonstrated fuller hysteresis curves and better energy dissipation capacity.

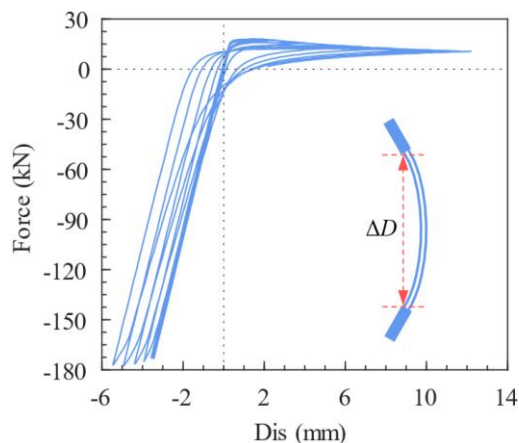


(a) Deformation of L1200-TestA

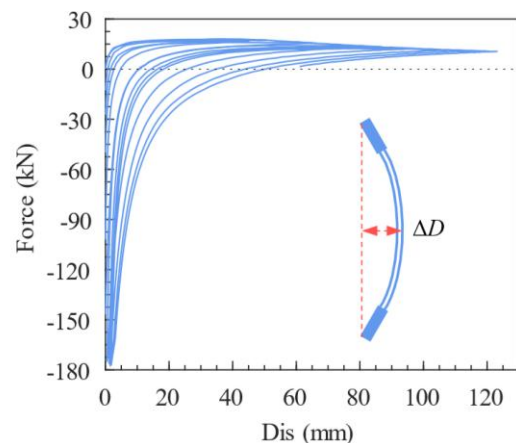


(b) Deformation of L500-TestB

Fig. 6 Deformation of specimens



(a) Vertical displacement of L1200-TestA



(b) Horizontal displacement of L1200-TestA

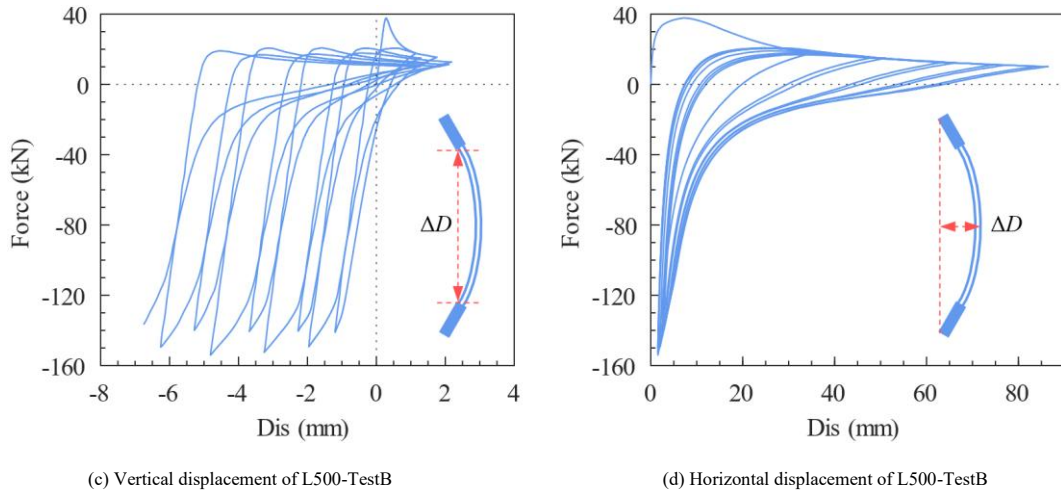


Fig. 7 The hysteresis curves of specimens

3. Numerical simulation

3.1. Finite element model

The FE model was developed using the general finite element software ABAQUS, with member dimensions consistent with the test specimens. Since the end connections are not the primary focus, bolts and bolt holes were assigned the same diameter to simplify computation. With reference to Article 12.2.4 of the Code for Design of Aluminum Structures [33], an initial imperfection amplitude of $L/1000$ was considered for the aluminum alloy members, where L represents the member length. This value was selected to reflect typical geometric imperfections observed in practical engineering applications, ensuring the model accurately captures the potential buckling behavior of the members. A buckling analysis was first conducted to determine the buckling mode of the member. The first mode exhibited a sine half-wave buckling around

the weak axis, which was used as the initial geometric imperfection distribution. This imperfection was applied by modifying the nodal coordinates of the elements.

The cyclic constitutive model of aluminum alloy was based on the Chaboche model, selected for its capability to accurately capture cyclic hardening and ratcheting effects under cyclic loading. Material parameters are defined in Table 3. Parameters σ_0 , Q_∞ and b represent isotropic hardening parameters, while C_k and γ_k denote kinematic hardening parameters. The FE model contained multiple contact pairs. The upper and lower ends and connecting plates were modeled as solids using C3D8R elements, an 8-node linear brick element with reduced integration suitable for contact analysis. The aluminum alloy members, being thin-walled structures, were modeled using S4R shell elements, a 4-node linear reduced integration shell element applicable to both thick and thin shells. The mesh distribution is shown in Fig. 8 (a), with uniform mesh division along the member's axial direction.

Table 3

Parameters of mixed hardening model

Parameter	σ_0	Q_∞	b	C_1	γ_1	C_2	γ_2	C_3	γ_3	C_4	γ_4
Value	233	28.7	4.8	8297	278	3501	146	3988	172	5194	190

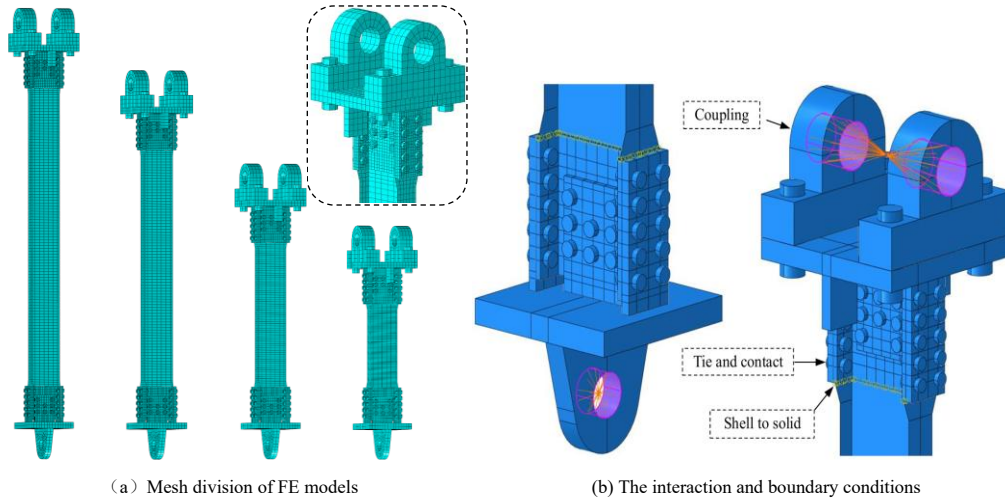


Fig. 8 FE models

Fig. 8(b) illustrates the interaction and boundary conditions. A reference point (RP) was set at the actuator shaft center on the upper end of the member, bound to two circular hole surfaces. The RP point had constrained translational degrees of freedom (DoFs) in the X and Y directions, rotation about the strong axis, and torsional DoF. Vertical translation and rotation about the weak axis were released. All contact surfaces among the actuator loading head, upper end, connecting bolts, upper and lower connecting plates, and aluminum alloy flanges were set to hard contact in the normal direction and frictionless in the tangential direction. Binding constraints were applied between connecting

plates and bolts, as well as between aluminum alloy flanges and bolts. The simplification assumes that the bolt diameter is equal to the hole diameter, which may underestimate the local stress concentration. However, the main purpose of this study is to evaluate the overall hysteretic performance and load-bearing capacity, so the simplification has limited impact on the overall results. The aluminum beam in the connecting section was modeled using solid elements, while the middle section used shell elements with shell-solid coupling. An RP point at the lower end hinge center was coupled with the hinge hole, constraining horizontal translation, rotation about the weak axis, and torsion

while releasing vertical translation. A nonlinear spring was set between the RP point and the ground to account for stiffness and slip caused by reaction frames, supports, and gaps.

3.2. Numerical simulation results

For each numerical model, four displacement peak points were selected to plot the principal stress (σ_1) distribution. The stress distribution at peak points for specimens L1200-TestA and L500-TestB is shown in Fig. 9(a) and (b). Gray regions indicate yielding under compression or tension. The stress distribution shows tensile and compressive yielding in the mid-span section after buckling under compression. With larger bending amplitudes, a plastic hinge forms at the mid-span, causing plastic deformation mainly through hinge rotation, while other areas remain elastic and recover upon unloading. Shorter members exhibit

a larger plastic region post-buckling. At tensile peak points, tensile yielding is observed in one flange, and full cross-sectional yielding may occur under sufficient tension. However, due to bolt-connected end holes reducing net cross-sectional bearing capacity, full cross-sectional tensile yielding is uncommon, generally only partially mitigating residual compressive deformation.

Experimental and simulated hysteresis curves of some specimens are compared in Fig. 9(c)-(f). The load-displacement curves from numerical simulations closely match the experimental results, with peak points and hysteresis shapes highly consistent. The maximum difference of peak point displacement and load value is less than 3%. The numerical results demonstrated that the adoption of shell elements combined with a mixed hardening material constitutive model achieved high computational accuracy, effectively capturing the stress states of structural members during the hysteretic loading process.

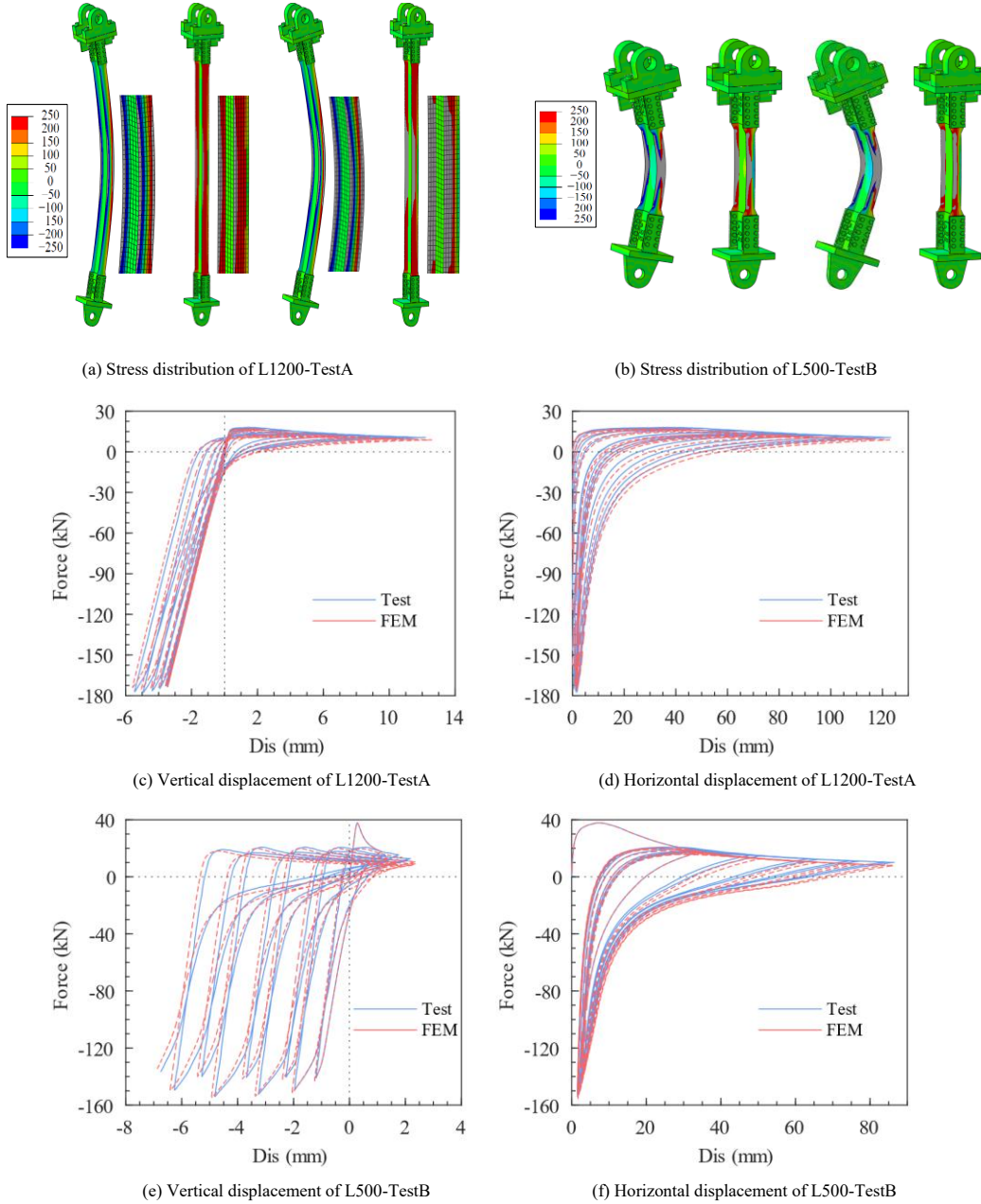


Fig. 9 Numerical simulation results

4. Parametric analysis

4.1. Finite element model for pin-ended member

In the hysteretic experiment, aluminum alloy members were connected to the testing apparatus using bolts and connecting plates. The stiffness of the end connectors was significantly higher than that of the members, making the test essentially a hysteretic loading of pin-ended members with rigid zones. To further study the hysteretic behavior of aluminum alloy members, a pin-ended

model was developed for parametric analysis, focusing on the effects of slenderness ratio, cross-sectional parameters, initial geometric imperfections, and tensile/compressive displacement amplitudes.

The pin-ended member FE model is shown in Fig. 10. RP points were set at the top and bottom centers, bound to the cross-section. The lower RP point released rotation about the weak axis while constraining other DoFs. The upper RP point released rotation about the weak axis and vertical translation while constraining other DoFs. The shell element type was S4R, chosen for its ability to accurately model both thick and thin shells while maintaining computational

efficiency, with material parameters consistent with the previous model. Buckling analysis was performed by applying a vertical concentrated load at the upper RP point. The first global buckling mode was used as the imperfection distribution. Material and geometric nonlinearity were then considered for the hysteretic analysis, with vertical displacement applied at the upper RP point.

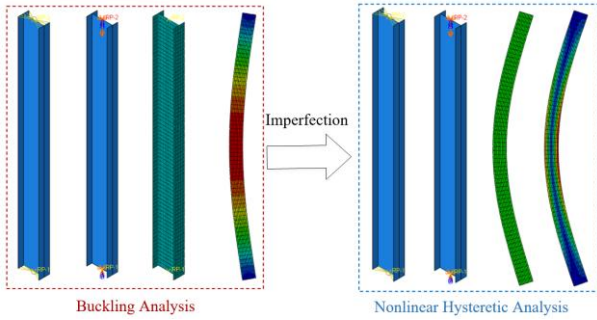


Fig. 10 FE models for pin-ended members

4.2. Effect of parameters

A FE model of pin-ended aluminum alloy members, as described previously, was established to perform a parametric analysis of hysteretic behavior. This study investigates the effects of geometric parameters and loading protocol parameters on hysteresis curves and stress distribution. Geometric parameters include slenderness ratio, cross-sectional parameters, and initial geometric imperfection. Loading protocol parameters encompass compressive displacement amplitude and tensile displacement amplitude. Each set of member parameters is shown in Table 4.

Table 4
Aluminum alloy member parameters

Slenderness Ratio	Cross-Section (mm)	Initial Imperfection Amplitude	Compressive Displacement Amplitude	Tensile Displacement Amplitude
50	H200×100×8×10	L/1000	0.01	-0.01
75	H200×100×8×10	L/1000	0.01	-0.01
100	H200×100×8×10	L/1000	0.01	-0.01
125	H200×100×8×10	L/1000	0.01	-0.01
150	H200×100×8×10	L/1000	0.01	-0.01
70	H200×60×8×10	L/1000	0.01	-0.01
70	H200×80×8×10	L/1000	0.01	-0.01
70	H200×100×8×10	L/1000	0.01	-0.01
70	H200×120×8×10	L/1000	0.01	-0.01
70	H200×140×8×10	L/1000	0.01	-0.01
70	H200×100×8×10	L/1000	0.01	-0.01
70	H200×100×8×10	L/500	0.01	-0.01
70	H200×100×8×10	L/200	0.01	-0.01
70	H200×100×8×10	L/100	0.01	-0.01
70	H200×100×8×10	L/50	0.01	-0.01
70	H200×100×8×10	L/1000	0.005	-0.01
70	H200×100×8×10	L/1000	0.0075	-0.01
70	H200×100×8×10	L/1000	0.01	-0.01
70	H200×100×8×10	L/1000	0.0125	-0.01
70	H200×100×8×10	L/1000	0.015	-0.01
70	H200×100×8×10	L/1000	0.005	-0.004
70	H200×100×8×10	L/1000	0.005	-0.006
70	H200×100×8×10	L/1000	0.005	-0.008
70	H200×100×8×10	L/1000	0.005	-0.010
70	H200×100×8×10	L/1000	0.005	-0.012

Hysteretic loading was applied in a single cycle, and the hysteretic

relationship between the vertical displacement at the loading point and the load was extracted. The hysteresis curves of the members under various parameters are shown in Fig. 11. Downward vertical displacement at the loading point was considered positive, while upward displacement was negative.

Both load and displacement were normalized. The vertical axis of the hysteresis curve was defined as P/A , where P is the load and A is the cross-sectional area. The horizontal axis was defined as δ/L , where δ is the displacement at the loading point and L is the member length.

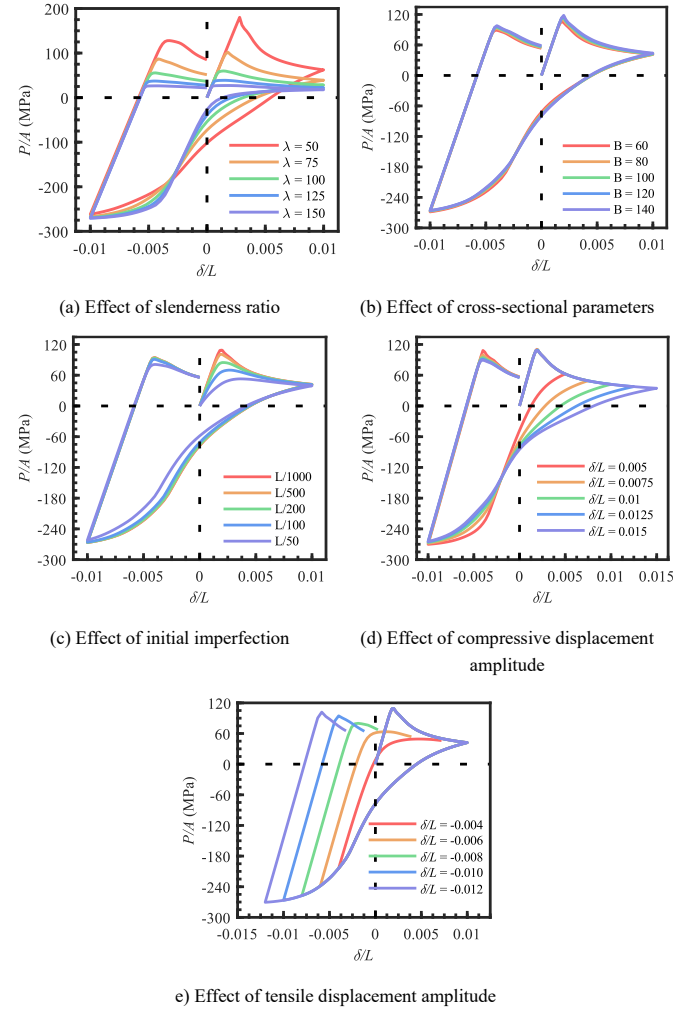


Fig. 11 Parametric analysis results

4.2.1. Effect of slenderness ratio

In the first set of models, the cross-section was H200×100×8×10, the initial geometric imperfection amplitude was $L/1000$, the compressive displacement amplitude was 0.01, and the tensile displacement amplitude was -0.01. The slenderness ratios were 50, 75, 100, 125, and 150.

As shown in Fig. 11(a), with other parameters constant, higher slenderness ratios correspond to lower stable bearing capacity. Longer members with high slenderness ratios exhibit a gradual decrease in bearing capacity after buckling, with negligible residual deformation after unloading, indicating primarily elastic deformation. Conversely, shorter members with low slenderness ratios show a rapid decline in bearing capacity after reaching the peak load and exhibit significant residual deformation upon unloading.

4.2.2. Effect of cross-sectional parameter

In the second set of models, the slenderness ratio was 70, the initial geometric imperfection amplitude was $L/1000$, and the compressive and tensile displacement amplitudes were 0.01 and -0.01, respectively. The cross-section was H200×B×8×10, with member widths B ranging from 60 mm to 140 mm. It is worth noting that if the member length remains constant while the beam width changes, the radius of gyration of the cross-section will also change, leading to variations in the slenderness ratio. Therefore, the member lengths need to be calculated based on cross-sectional parameters to ensure a slenderness ratio of 70. As shown in Fig. 11(b), the hysteresis curves nearly overlap, indicating that cross-sectional parameters have negligible influence on hysteretic behavior.

This is mainly because, under a fixed slenderness ratio, the global buckling

mode dominates the deformation process of H-section aluminum alloy members. In such cases, variations in sectional width do not significantly affect the overall stability or hysteretic response. However, this conclusion is valid only within the investigated range of slenderness ratios and loading conditions, and may not hold in cases where local buckling or interaction between local and global modes becomes critical.

4.2.3. Effect of initial geometric imperfection

In the third set of models, the slenderness ratio was 70, the cross-section was $H200 \times 100 \times 8 \times 10$, the compressive displacement amplitude was 0.01, and the tensile displacement amplitude was -0.01. Initial geometric imperfection amplitudes were $L/1000$, $L/500$, $L/200$, $L/100$, and $L/50$.

As shown in Fig. 11(c), larger initial imperfection amplitudes intensify geometric nonlinearity during compression, reducing the stable bearing capacity. For members with larger imperfections, tangent stiffness decreases gradually during compression, with a smoother post-peak decline. In contrast, smaller imperfections exhibit sudden buckling with a rapid post-peak capacity drop.

4.2.4. Effect of compressive displacement amplitude

In the fourth set of models, the slenderness ratio was 70, the cross-section was $H200 \times 100 \times 8 \times 10$, the initial geometric imperfection amplitude was $L/1000$, and the tensile displacement amplitude was -0.01. Compressive displacement amplitudes ranged from 0.005 to 0.015.

The effect of the compressive displacement amplitude on the hysteresis curves is shown in Fig. 11(d). Due to consistent geometric parameters, the hysteresis curves completely overlapped before buckling occurred. As the compressive displacement amplitude increased, the bending deformation became more pronounced, resulting in larger residual deformation after unloading. During the tensile phase, the hysteresis curves of all members converged at a fixed point, but the slope of the hysteresis curves passing through this point varied.

4.2.5. Effect of tensile displacement amplitude

In the fifth set of models, the slenderness ratio was 70, the cross-section was $H200 \times 100 \times 8 \times 10$, the initial geometric imperfection amplitude was $L/1000$, and the compressive displacement amplitude was 0.01. Tensile displacement amplitudes ranged from -0.004 to -0.012. As shown in Fig. 11(e), the compression process remains consistent across amplitudes. Smaller tensile amplitudes result in less plastic deformation correction and larger residual deformation, corresponds to lower stable bearing capacity upon reloading. The stable bearing capacity of the member with a tensile displacement amplitude of

-0.004 was significantly lower during reloading compared to the member with an amplitude of -0.012.

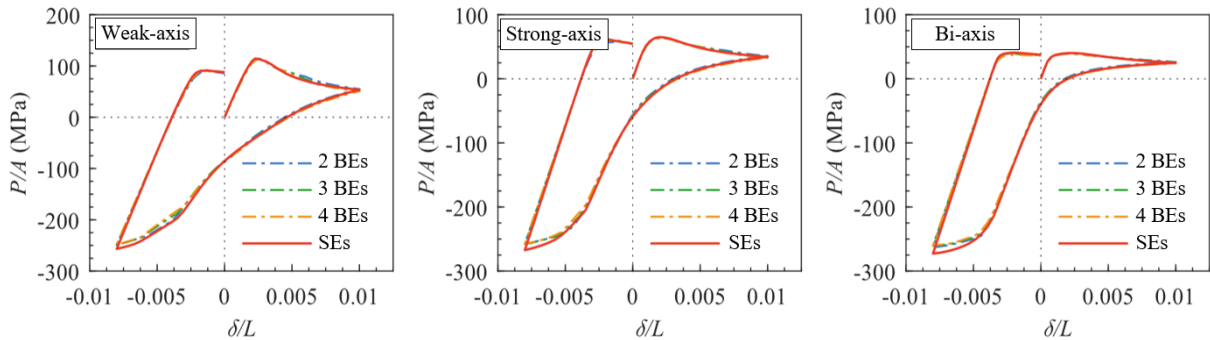
5. Simulation methods for aluminum alloy latticed shell member

5.1. Numerical simulation of independent member

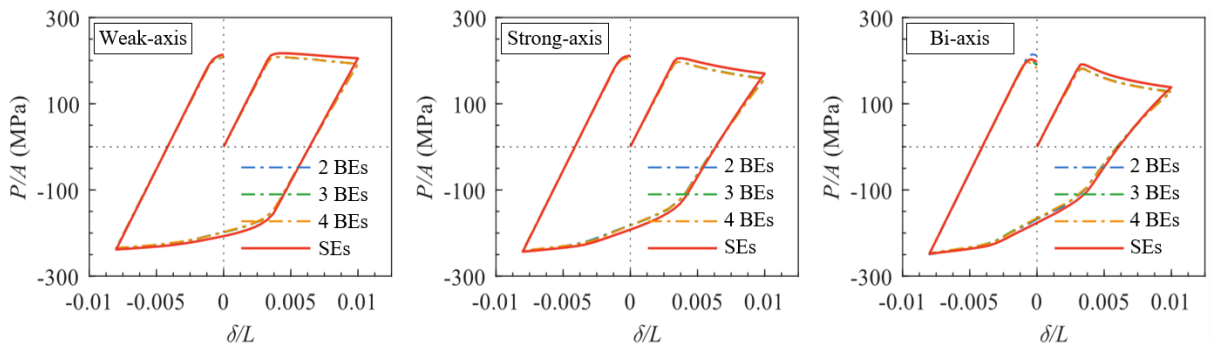
In aluminum alloy latticed shells, the large number of members and their complex stress states result in multiple buckling modes under loading. Finite element method numerical models can better simulate the hysteretic behavior of these members. While solid and shell elements provide high computational accuracy, their efficiency is low and unsuitable for large latticed shell structures. Dividing members into multiple beam elements allows the simulation of both compressive buckling and tensile yielding.

Aluminum alloy members often feature an I-shaped cross-section, categorized as open thin-walled sections. Elements based on cross-sectional warping theory are more suitable for such profiles. Subsequent seismic response analysis of latticed shells was conducted using the ABAQUS finite element software, which offered various beam element types. Among them, the B320S beam element is a three-dimensional beam element considering shear deformation and warping effects, making it ideal for simulating open thin-walled sections.

This section investigated the numerical accuracy of the B320S beam element to derive an appropriate member simulation method. The numerical accuracy of the shell element had been experimentally validated and was thus used as a benchmark to evaluate the beam element's accuracy. Three loading methods were considered: weak-axis eccentric loading, strong-axis eccentric loading, and bi-axial eccentric loading. The member cross-section was $H200 \times 100 \times 8 \times 10$, with slenderness ratios ranging from 50 to 120 about the weak axis. Boundary conditions at both ends were pin-ended along the strong and weak axes. The eccentric distances were 5 mm for the weak axis, 20 mm for the strong axis, and 5 mm and 20 mm for the weak and strong axes, respectively, in bi-axial loading. The members underwent a compression-first then tension loading protocol with maximum compressive displacement amplitude of 0.01 and tensile displacement amplitude of -0.08 under displacement loading. Partial hysteretic loading results are shown in Fig. 12, with subFig.s (a), (b), and (c) representing slenderness ratios of 60, 90, and 120, respectively. According to the hysteresis curves obtained from the shell and beam elements, the simulation accuracy for weak-axis, strong-axis, and bi-axial eccentric loading was high. The bearing capacity obtained from the shell element was slightly higher than that from the beam element, but the overall difference was minimal.



(a) Member slenderness ratio $\lambda = 60$



(b) Member slenderness ratio $\lambda = 90$

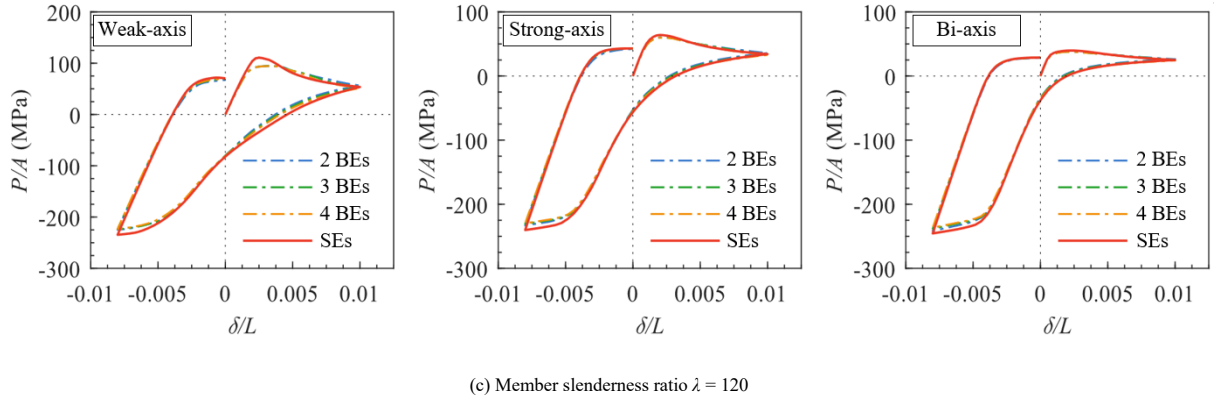


Fig. 12 Comparison of hysteresis curves between segmented beam elements and shell elements

Note: Weak-axis, Strong-axis, and Bi-axis denote weak-axis eccentric loading, strong-axis eccentric loading, and bi-axial eccentric loading, respectively. 2BEs, 3BEs, 4BEs represent segmented beam elements with 2, 3, and 4 elements, and SEs stands for shell elements.

Fig. 13 shows the impact of the number of member segments on computational accuracy. SubFigs (a), (b), and (c) correspond to 2, 3, and 4 segments, respectively. The hysteretic loop area reflects the energy dissipation capacity of the members and serves as a comprehensive indicator of computational accuracy. The horizontal axis represents the hysteretic loop area calculated using beam elements, while the vertical axis represents the hysteretic loop area from shell elements. To facilitate comparison, the hysteretic loop area is normalized. The diagonal solid line indicates identical results from both element types; points above the line indicate higher results from shell elements,

while points below suggest the opposite. The dashed lines represent a 5% error margin, with points within this range indicating less than 5% difference in results. As can be seen from the figure, almost all data points fall within the dashed line, indicating that the numerical accuracy of beam elements is high, with hysteretic loop area differences mostly within 5%. Moreover, as the B320S element is a second-order beam element with high accuracy, the number of beam segments has minimal impact on results. Thus, when constructing the FE model of latticed shells, members can be divided into four beam elements.

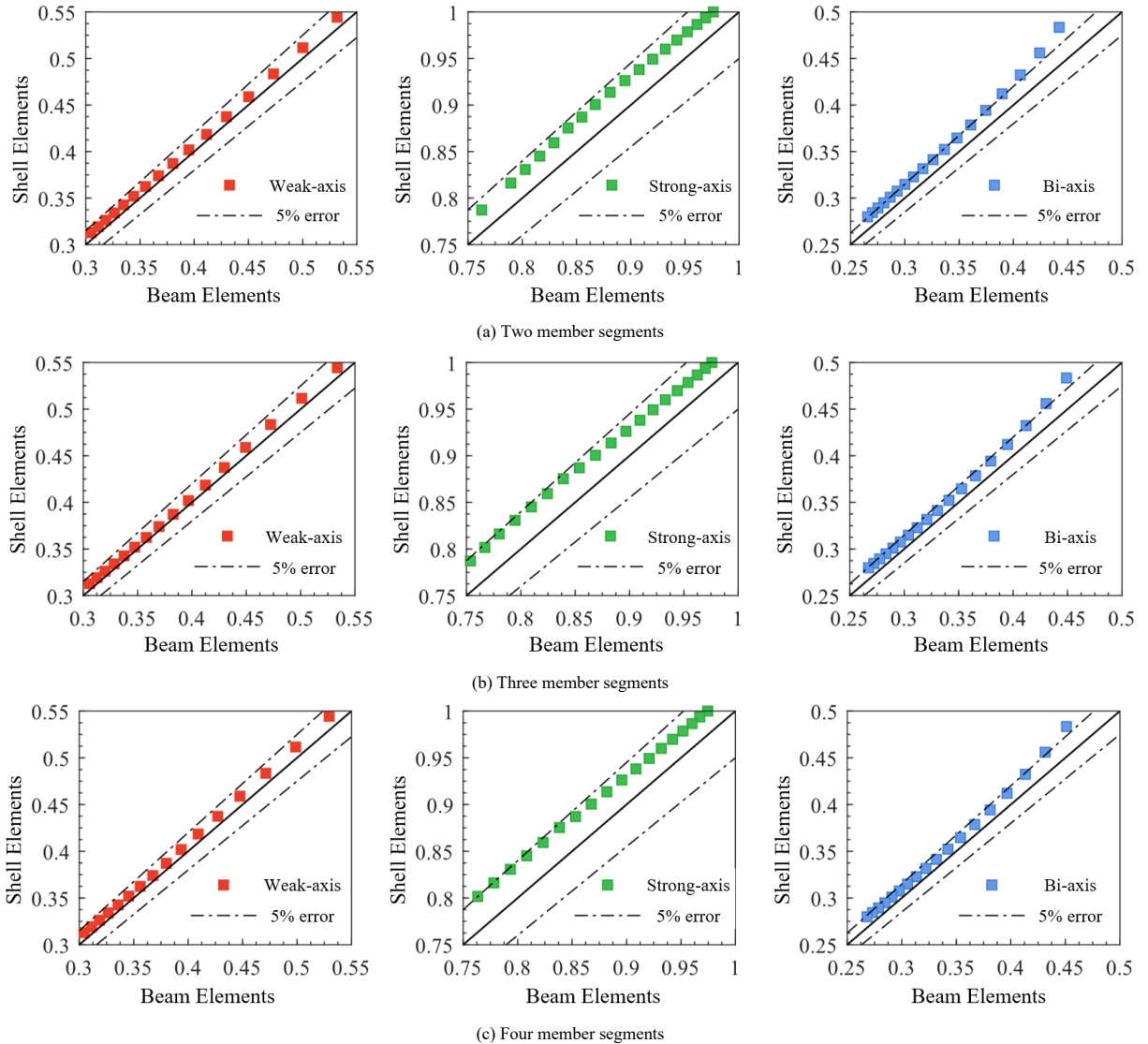


Fig. 13 Impact of the number of member segments on computational accuracy

5.2. Numerical simulation of latticed shell member

The previous analysis showed that segmented beam elements produced results similar to shell elements, effectively simulating the hysteretic loading process. However, the boundary conditions in the prior study were pin-ended, differing from those of latticed shell members. Additionally, under seismic action, the stress state of latticed shell members is complex, with significant tension-compression asymmetry and bending moments and shear forces in both the strong and weak axes.

To further evaluate the simulation accuracy of beam elements for latticed shell members, a multi-scale latticed shell model was established, as shown in Fig. 14. Some members were simulated using shell elements, while the rest used beam elements with four segments per member. A full beam element latticed

shell model was also constructed, and the axial displacement time-history curves of corresponding members were compared, as shown in Fig. 15. The displacement time-history curves obtained from both element types closely overlapped, indicating that the multi-segment beam element simulation results were reliable.

In addition to accuracy, the computational efficiency of the two models was also compared. The shell model contains more than 200% of the degrees of freedom of the beam model and requires approximately 2.5 times the computation time for the same analysis case. Since the beam model achieves comparable accuracy in simulating global hysteretic behavior while maintaining much lower computational cost, it is more suitable for large-scale structural analyses.

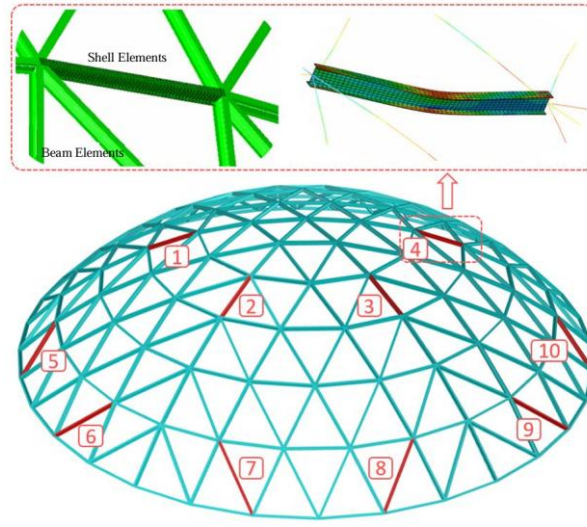


Fig. 14 Multi-scale latticed shell model

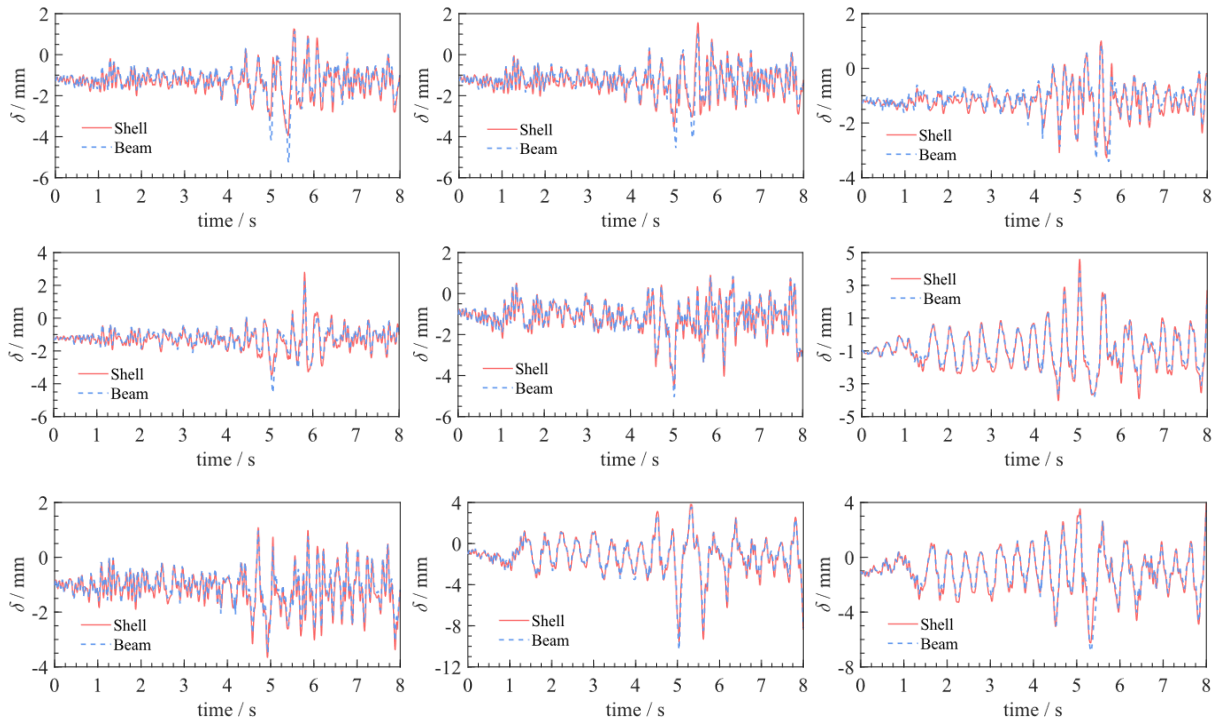


Fig. 15 Comparison of latticed shell member simulation results

6. Conclusions

This study investigated the hysteretic behavior of H-shaped aluminum alloy members commonly used in aluminum alloy gusset joint latticed shells. By combining experimental loading and numerical simulation methods, actual hysteresis curve data of aluminum alloy members were obtained, a refined FE model was established, and the impact of key parameters on the hysteretic

loading process was analyzed. Finally, the applicability of beam elements in multi-scale latticed shell model was evaluated. The main conclusions are as follows:

(1) Hysteretic tests revealed that under cyclic loading, aluminum alloy members exhibit significant bearing capacity degradation after compression buckling. Tensile loading partially recovers residual deformation. Slenderness ratio significantly affects hysteretic behavior, with higher slenderness ratios

leading to more severe post-buckling bearing capacity degradation.

(2) Numerical results demonstrated that FE model employing shell elements and mixed hardening material constitutive models achieved high computational accuracy, accurately simulating the buckling process and hysteretic behavior of members, thus verifying the reliability of the numerical method.

(3) Parametric analysis indicated that slenderness ratio is a key parameter influencing hysteretic behavior. A higher slenderness ratio results in more pronounced post-buckling bearing capacity degradation. Increased initial geometric imperfection reduces stable bearing capacity. Larger compressive displacement amplitudes expand residual deformation and reduce stable bearing capacity upon reloading. Larger tensile amplitudes result in more plastic deformation correction.

(4) The multi-scale latticed shell model analysis showed that using a four-segment B320S beam element effectively simulates the hysteretic behavior of members in latticed shells, with computational accuracy differing by less than 5% from shell element results.

This study fills the gap in hysteretic behavior research of aluminum alloy members under overall buckling conditions, providing essential experimental data and validated numerical analysis methods to support seismic design of aluminum alloy latticed shell structures.

Acknowledgements

This work was supported by the National Natural Science Foundation of China (grant number No. 51478335).

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STUDY ON POST-FIRE MECHANICAL PERFORMANCE OF Q460QENH WEATHERING STEEL WITH THREE COOLING METHODS

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ABSTRACT

Monotonic tensile tests were performed to investigate the mechanical behavior of Q460qENH weathering steel after exposure to temperatures ranging from 20°C to 1000°C, considering three different cooling methods: cooling in air (CIA), cooling in water (CIW), and cooling in fire-fighting foam (CIF). The tensile fracture surfaces were analyzed using scanning electron microscopy (SEM), and the post-fire mechanical properties were systematically evaluated. The results revealed that the fracture surfaces under CIA and CIF conditions exhibited similar dimple morphologies, while the number of dimples decreased progressively with increasing temperature under CIW conditions. Both the cooling method and the exposure temperature exerted only a limited effect on the elastic modulus. A distinct yield plateau in the stress - strain curve disappeared under all cooling conditions after exposure to temperatures of 800 °C or higher. Under CIA and CIF conditions, the yield strength decreased markedly after exposure to temperatures above 600 °C, while the ultimate tensile strength remained relatively stable. In contrast, under CIW conditions, the yield strength initially declined but recovered to nearly its original value after exposure to 700 °C, accompanied by a pronounced increase in ultimate strength. The CIW process was found to reduce ductility but enhance Brinell hardness. Furthermore, a mathematical model was developed to describe the post-fire mechanical properties under different cooling conditions, providing theoretical support for assessing the residual load-bearing capacity of steel structures following fire exposure.

ARTICLE HISTORY

Received: 27 March 2025
Revised: 23 August 2025
Accepted: 23 August 2025

KEYWORDS

Q460qENH weathering steel;
Cooling methods;
Monotonic tensile;
Post-fire mechanical properties;
Mathematical model

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1. Introduction

Weathering steel is a high-performance structural steel commonly used in bridge construction without the need for additional protective coatings. However, weathering steel bridges may be exposed to extreme events during their service life, such as fires caused by traffic accidents, spontaneous vehicle combustion, or electrical failures [1-2]. The unpredictable nature of these incidents, combined with the difficulties associated with firefighting, can result in severe damage to critical structural components, potentially leading to the decommissioning or even demolition of entire bridges [3-5]. Existing design codes for steel structures primarily focus on fire protection in buildings. Therefore, understanding the post-fire mechanical properties of weathering steel is essential for ensuring the safety and reliability of high-performance weathering steel bridges, and has consequently become a key topic in recent research.

Studies have shown that the post-fire mechanical properties of different steels are strongly affected by factors such as manufacturing processes, steel grades, and fire exposure conditions. Song [6] investigated the influence of thermomechanical controlled processing (TMCP) and quenching - tempering (QT) treatments on the post-fire mechanical properties of high-strength structural steels. Wang [7] examined the effects of fire exposure temperature, cooling methods, heating rates, and initial loading conditions on the post-fire mechanical performance of Q690 high-strength steel. The results indicated that high heating rates, repeated heating and cooling cycles, and axial loading conditions significantly reduced both the elastic modulus and yield strength. Qiang [8] studied the post-fire performance of S960 ultra-high-strength steel, revealing substantial differences compared to S460, S690, and low-carbon steels. The study identified 600 °C as the phase transformation temperature for S960 steel and proposed an empirical model to predict its post-fire mechanical properties based on experimental data. Li [9] analyzed the effect of heating rates on the post-fire mechanical behavior of high-strength steels and concluded that the heating rate had minimal influence on their performance. Ren [10] investigated the post-fire mechanical properties of Q235 cold-formed steel sections, finding that member thickness had a negligible effect, whereas notable differences in mechanical performance were observed between flat and curved regions.

The influence of fire extinguishing methods on the material properties of steel structures is also of considerable importance. Notable variations in post-fire mechanical properties have been observed among different steel grades [11-13]. Zhang [11] investigated the effects of foam and water immersion cooling

on the post-fire mechanical behavior of Q355 steel, reporting that when the exposure temperature exceeded 600 °C, the yield strength and ultimate tensile strength of specimens decreased under CIF conditions, whereas those under CIW conditions increased significantly. Zhou [12] examined the mechanical properties of Q620 high-strength steel under CIA, CIW, and CIF conditions, observing minimal differences between CIA and CIF, but substantial disparities between CIW and the other two cooling methods. Zhang [13] studied the post-fire mechanical behavior of Q345 structural steel subjected to repeated heating and cooling cycles between 200 °C and 900 °C under CIF conditions. With an increasing number of thermal cycles, pronounced degradation was observed in yield strength, ultimate tensile strength, and fracture elongation.

Q460qENH steel is a widely used high-strength steel developed in recent years for bridge construction in China. It has a minimum yield strength of 460 MPa. The letter “E” represents the quality grade, indicating that the steel satisfies the requirements of the -40 °C impact test, thereby ensuring good toughness even in low-temperature environments. The letters “NH” signifies the steel’s excellent resistance to atmospheric corrosion. However, research on the post-fire mechanical properties of Q460qENH weathering steel - particularly its mechanical behavior after exposure under different cooling methods - remains limited.

In the present study, experimental investigations were carried out to examine the post-fire mechanical behavior of Q460qENH weathering steel under three different cooling conditions. The microscopic damage mechanisms of the tensile fracture surfaces were analyzed using scanning electron microscopy (SEM), and the effects of fire exposure temperature and cooling method on the mechanical properties were systematically evaluated. Based on the experimental results, a mathematical model was developed to describe the relationship between mechanical behavior and fire exposure temperature under various cooling conditions. Furthermore, the post-fire mechanical properties of structural steels reported in previous studies were compiled and compared with the experimental data obtained in this work.

2. Test investigation

2.1. Materials and coupons

The Q460qENH weathering steel plate used in this study had a thickness of 10 mm and was manufactured by Nanjing Iron and Steel Co., Ltd. The chemical composition of the Q460qENH steel is presented in Table 1.

Table 1

The chemical composition of Q460qENH weathering steel (wt/%)

Chemical composition	C	Si	Mn	P	S	Ti	Cu	Cr	Ni
Content	0.07	0.26	1.4	0.012	0.002	0.012	0.29	0.48	0.29

The specimens were prepared in accordance with the GB/T 228.1 - 2021 standard [14]. The tensile loading direction was oriented perpendicular to the rolling direction of the steel plate. The geometry and dimensions of the specimens are illustrated in Fig. 1.

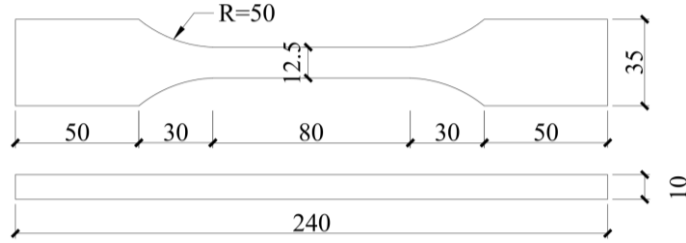


Fig. 1 Geometry of the specimens (mm)

2.2. Test setup

The experimental program consisted of three stages: heating, cooling, and room-temperature testing. The YTH-12-12 integrated muffle furnace was employed during the heating stage, as shown in Fig. 2(a). Nine specimens from each group were heated to target temperatures of 200 °C, 400 °C, 500 °C, 600 °C, 700 °C, 800 °C, 900 °C, and 1000 °C at a constant heating rate of 15 °C/min. The specimens were maintained at each target temperature for 30 minutes to ensure uniform thermal distribution. To investigate the effect of different cooling methods on the post-fire mechanical properties of Q460qENH weathering steel, the specimens were subjected to one of three cooling conditions immediately after removal from the furnace, as illustrated in Fig. 2. For the cooling in air (CIA) condition, specimens were placed on a stone platform and allowed to cool naturally to ambient temperature (approximately 25 °C). For the cooling in fire-fighting foam (CIF) condition, specimens were transferred to an open container, where aqueous film-forming foam (AFFF) was continuously sprayed until the foam completely submerged the specimens and no visible vapor remained. The foam-covered specimens were then left to cool beneath the insulating layer. For the cooling in water (CIW) condition, specimens were directly immersed in a water-filled container, removed upon reaching room temperature, and gently wiped dry.



(a) Integrated muffle furnace



(b) CIA



(c) CIW



(d) CIF

Fig. 2 Heating and cooling methods

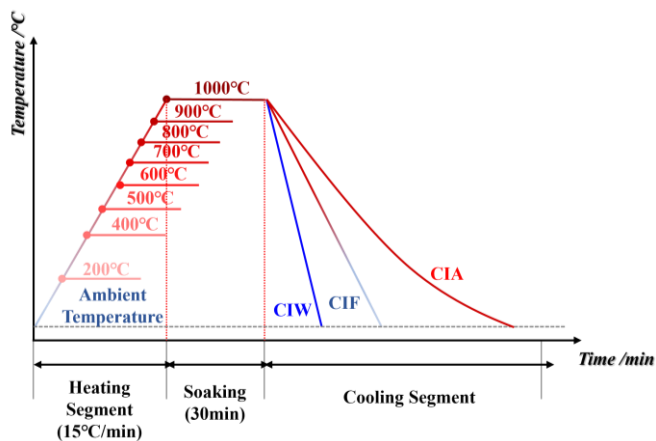


Fig. 3 Specimens heating and cooling curve

The cooling behavior associated with each method was also investigated. The time required for the specimens to cool from the target temperature to room temperature was recorded, and the average cooling rates for the CIA, CIF, and CIW conditions were approximately 1 °C/s, 5 °C/s, and 100 °C/s, respectively. It should be noted that these values represent average rates over the entire cooling process. In practice, the temperature evolution during cooling exhibits a nonlinear trend - characterized by a rapid initial decrease followed by a slower decline - consistent with the exponential decay patterns illustrated in Fig. 3.

To ensure the representativeness of the experimental results, three identical specimens were tested for each combination of target temperature (ranging from 200 °C to 1000 °C) and cooling method, resulting in a total of 72 high-temperature specimens (eight temperature levels and three cooling methods), along with three control specimens tested at room temperature (25 °C). Following the heating and cooling phases, monotonic tensile tests were performed at room temperature using a SANS SHT-600KN electro-hydraulic servo testing machine. The tests were conducted under displacement control at a loading rate of 1.5 mm/min, in accordance with the GB/T 228.1 - 2021 standard

[14]. Each test was continued until final fracture occurred. Tensile strain within the gauge section was measured using an 80 mm extensometer, and the corresponding data were recorded by the DCS - 200 data acquisition system. In addition, the effects of heating temperature and cooling method on the material

hardness were evaluated through Brinell hardness measurements, performed using an HB-3000B testing instrument in compliance with GB/T 231.1 - 2018 [15]. The experimental setups are illustrated in Fig. 4.



Fig. 4 Experimental setup

3. Test results

3.1. Visual observations

The post-fire surface appearance of Q460qENH weathering steel under the three cooling methods is illustrated in Fig. 5. For CIA specimens, a dark black oxide layer formed on the surface when the exposure temperature exceeded

600 °C, accompanied by partial spalling of the oxide layer. Correspondingly, the elongation after fracture increased. CIF specimens exhibited a tan surface color at exposure temperatures below 600 °C. As the temperature increased beyond 600 °C, the surface color gradually darkened to charcoal black. In these cases, the elongation after fracture showed minimal variation, except for the CIW1000 specimen. CIW specimens displayed a similar trend in surface color evolution to that observed in the CIF specimens.

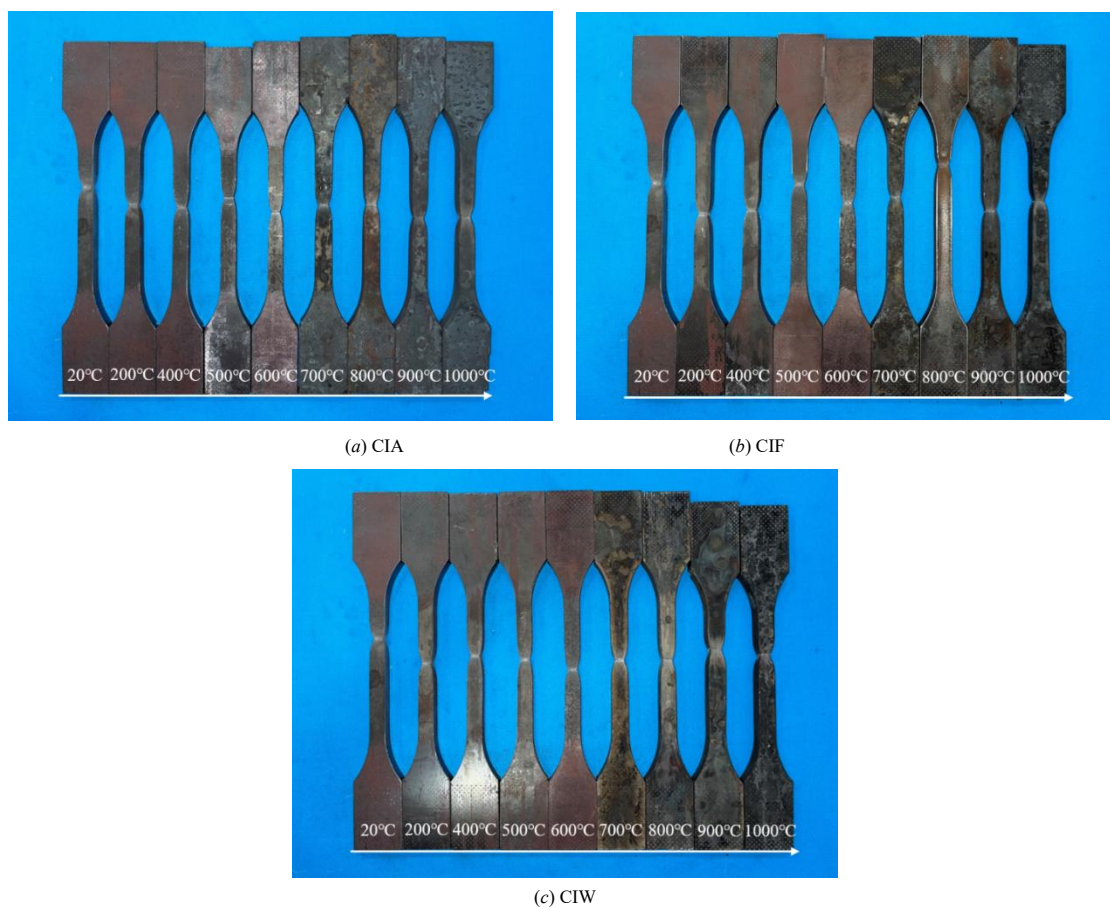


Fig. 5 Surface feature and failure patterns of specimens

The foam cooling method employed in this study utilized a 3% AFFF fire suppressant, which is a water-based agent primarily composed of fluorocarbon surfactants, hydrocarbon surfactants, solvents (e.g., glycol ethers), and foam

stabilizers [16]. Upon contact with a high-temperature surface, the water component in the foam rapidly evaporates, resulting in intense localized heat absorption during the early stage of cooling. As cooling progresses, the foam

forms a stable surface layer that significantly reduces the evaporation rate and insulates the specimen from ambient air. At this stage, heat dissipation occurs predominantly through conduction within the foam layer, while convective cooling is largely suppressed [17]. Consequently, the overall cooling efficiency gradually decreases, and the average cooling rate of CIF specimens over the full cooling period becomes comparable to that of CIA specimens, despite the higher initial heat extraction rate. This mechanism accounts for the similar mechanical properties observed under CIF and CIA conditions. It should also be noted that AFFF functions through a physical cooling process and does not chemically react with metallic materials such as steel during high-temperature exposure [18]. As shown in Fig. 5(b), no secondary chemical products resulting from interactions between the foam and the steel surface were observed.

3.2 Microfracture characteristics

The microfracture characteristics of specimens subjected to three different cooling methods were examined using a scanning electron microscope (SEM), as shown in Fig. 6. All fracture surfaces exhibited typical microvoid coalescence features, characterized by dimples of varying quantity, size, and depth. These

dimples originated as microscopic voids formed during localized plastic deformation, with the fracture process comprising stages of void nucleation, growth, and coalescence. The consistent presence of dimple patterns is a fundamental indicator of ductile fracture mechanisms, and their areal fraction correlates directly with the material's ductility [19].

At the lower exposure temperature of 600 °C, the microfracture morphology exhibited only minor variations among the three cooling regimes. However, at elevated temperatures of 800 °C and 1000 °C, the influence of the cooling method became more pronounced, with distinct morphological features emerging under each condition. The CIA800 specimens displayed a dense distribution of deep dimples, indicating superior ductility. In contrast, the CIW800 specimens exhibited shallower and more sparsely distributed dimples, suggesting reduced plasticity. Similarly, the CIA1000 specimens revealed numerous dimples of varying sizes, whereas the CIF1000 specimens showed fewer dimples and a more uniform fracture surface. Notably, the CIW1000 specimens demonstrated a marked reduction in both the number and size of dimples, along with a smoother fracture surface, reflecting the pronounced effect of rapid water cooling on ductility degradation.

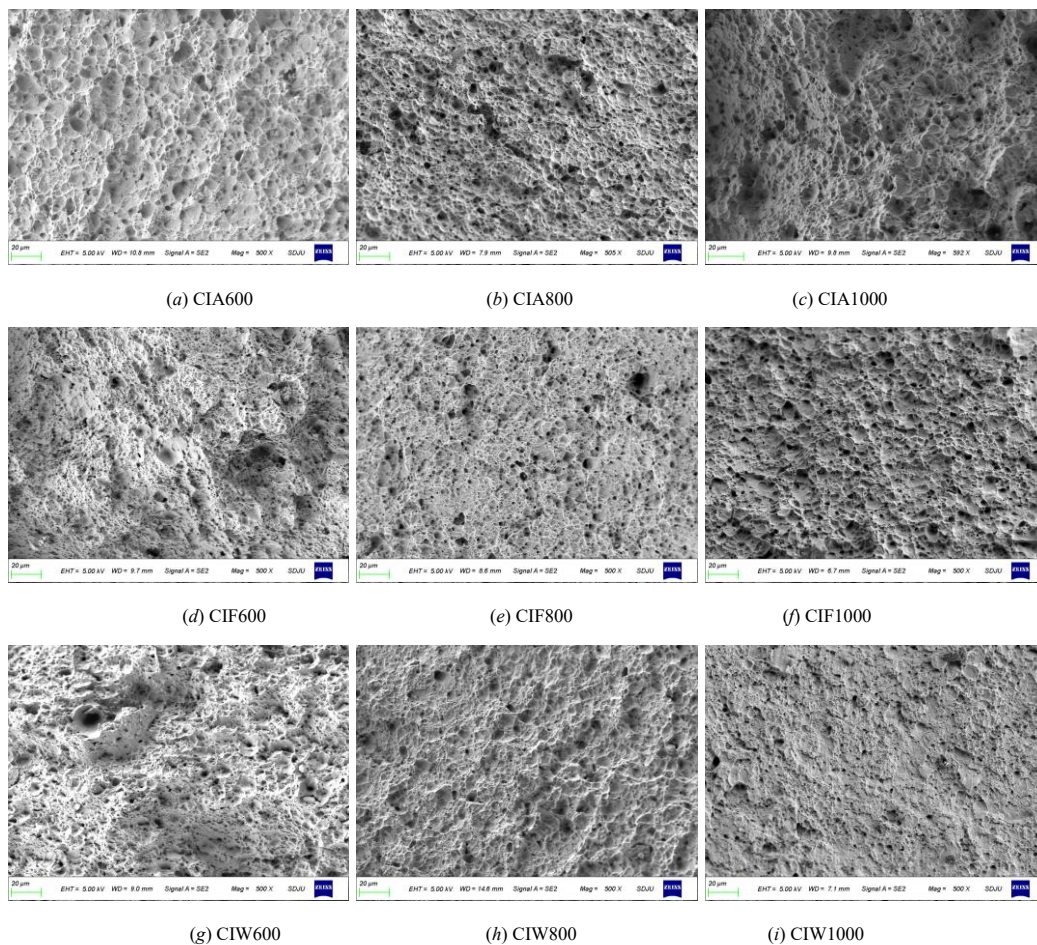


Fig. 6 Microscopic fracture morphology

Although direct metallographic observations were not conducted in this study, it is well established that higher cooling rates promote the transformation of austenite into martensite, which enhances strength but reduces ductility [20]. In contrast, moderate cooling rates may lead to the formation of bainitic structures, while slower cooling facilitates the development of pearlite or a ferrite-pearlite mixture [21]. Furthermore, the observed evolution of fracture surface features qualitatively correlates with the temperature-dependent variation in elongation after fracture. For instance, specimens such as CIW800 and CIW1000, which exhibited smoother fracture surfaces with sparse and shallow dimples, also recorded significantly lower η_{ET} values. In comparison, the CIA800 and CIF800 specimens displayed denser and deeper dimples, consistent with their higher ductility. This qualitative agreement reinforces the established relationship between microscopic dimple morphology and macroscopic deformation capacity, as widely documented in ductile fracture studies [10,11,13,19].

4. Post-fire mechanical properties

4.1. Stress-strain curves

The stress - strain curves of Q460qENH weathering steel subjected to three different cooling methods after fire exposure are presented in Fig. 7.

Fig. 7 illustrates that at exposure temperatures up to 700 °C, the stress - strain curves of post-fire specimens remain closely aligned with those obtained under ambient conditions, exhibiting typical ductile failure behavior. Notably, the CIA700 and CIF700 specimens exhibit a pronounced reduction in yield strength compared to other cases, whereas the CIW700 specimens maintain a relatively stable yield strength. Q460qENH weathering steel displays a distinct yield plateau at lower temperatures (e.g., from room temperature to 600 °C), indicative of yield point elongation associated with strain aging and coordinated dislocation motion. However, as the exposure temperature increases - particularly beyond 800 °C - the yield plateau that is clearly visible at lower temperatures progressively disappears, and the material transitions from discontinuous to continuous yielding. This transformation reflects the microstructural reconstruction induced by high temperatures, including static

recrystallization, dissolution of precipitates, and a reduction in dislocation density, which collectively diminish the yield delay mechanism [22,23]. Similar yielding behavior has been reported in other high-strength steels such as β -Ti alloy and Ti-5553 alloy, where the disappearance of the yield plateau serves as a useful indicator of thermal degradation and the evolution of deformation mechanisms [24,25]. While the ultimate strength remains largely unchanged for CIA and CIF specimens, a significant increase is observed in CIW specimens. These findings suggest that the mechanical properties of Q460qENH weathering steel undergo a transitional shift within the fire exposure temperature range of 700 °C to 800 °C, with the cooling method exerting a substantial influence on the nature of this mechanical transition.

Tables 2-4 summarize the post-fire mechanical properties of Q460qENH weathering steel under varying temperatures and cooling methods. A factor (η) is introduced to quantify the relative changes in mechanical parameters after high-temperature exposure. In Tables 2-4, η_E , η_{fy} , η_{fu} , η_{ET} and η_{EH} represent the ratios of post-fire test results to room-temperature test results for elastic modulus, yield strength, ultimate strength, elongation after fracture, and Brinell hardness, respectively. For stress-strain curves exhibiting a distinct yield plateau, the yield strength was defined as the lower yield point, and for curves without a yielding platform, the nominal yield strength was determined as the stress corresponding to 0.2% plastic strain.

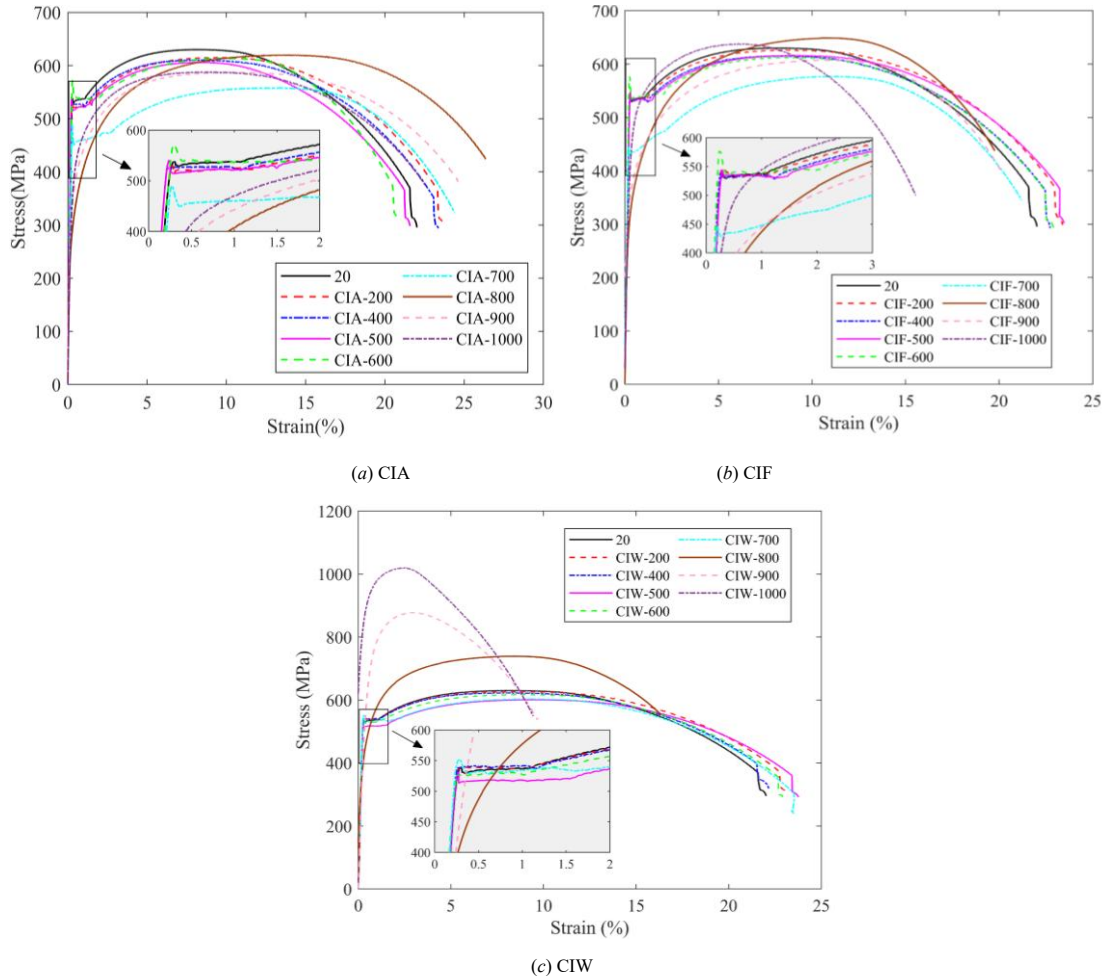


Fig. 7 Stress-strain relationships

Table 2
CIA test results (mean value)

Temperature (°C)	Elastic Modulus		Yield Strength		Ultimate Strength		Elongation after Fracture		Brinell Hardness	
	$E/(GPa)$	η_E	$f_y/(MPa)$	η_{fy}	$f_u/(MPa)$	η_{fu}	$\varepsilon_T/(\%)$	$\eta_{\varepsilon T}$	$E_H/(HB)$	η_{EH}
20	220.10 (11.63)	1.00	524.4 (11.16)	1.00	625.9 (9.39)	1.00	8.1 (0.06%)	1.00	194 (1.63)	1.00
200	222.20 (10.76)	1.01	518.9 (4.30)	0.99	619.8 (3.24)	0.99	7.9 (0.02%)	0.98	187 (1.41)	0.96
400	212.80 (10.09)	0.97	523.5 (4.68)	1.00	612.3 (2.48)	0.98	8.2 (0.31%)	1.01	198 (0.82)	1.02
500	217.65 (15.23)	0.99	513.5 (11.77)	0.98	603.6 (11.37)	0.96	7.5 (0.45%)	0.93	184 (1.41)	0.95
600	215.35 (11.18)	0.98	534.5 (2.86)	1.02	612.3 (6.21)	0.98	8.4 (0.67%)	1.04	182 (2.62)	0.94
700	208.15 (13.33)	0.95	444.0 (17.38)	0.85	579.9 (17.38)	0.93	9.3 (0.42%)	1.15	187 (0.82)	0.96
800	210.10 (11.40)	0.95	278.5 (2.78)	0.53	623.2 (6.09)	1.00	10.1 (0.08%)	1.25	180 (1.70)	0.93
900	227.90 (18.53)	1.04	312.1 (7.56)	0.60	588.6 (6.13)	0.94	9.5 (0.19%)	1.17	183 (2.05)	0.94
1000	205.65 (18.54)	0.93	347.8 (13.19)	0.66	587.9 (0.12)	0.94	8.1 (0.18%)	1.00	171 (4.19)	0.88

Note: The values in parentheses represent the standard deviation of the corresponding results

Table 3
CIF test results (mean value)

Temperature (°C)	Elastic Modulus		Yield Strength		Ultimate Strength		Elongation after Fracture		Brinell Hardness	
	$E/(GPa)$	η_E	$f_y/(MPa)$	η_{fy}	$f_u/(MPa)$	η_{fu}	$\varepsilon_T(\%)$	$\eta_{\varepsilon T}$	$E_H/(HB)$	η_{EH}
20	220.10 (11.64)	1.00	524.4 (11.16)	1.00	625.9 (9.39)	1.00	8.1 (0.06)	1.00	194 (1.63)	1.00
200	218.53 (7.58)	0.99	526.0 (8.95)	1.00	621.3 (9.35)	0.99	8.5 (1.00%)	1.05	186 (2.94)	0.96
400	214.50 (3.96)	0.97	524.7 (4.96)	1.00	613.9 (4.75)	0.98	7.9 (0.34%)	0.98	196 (3.30)	1.01
500	217.65 (7.75)	0.99	518.3 (9.62)	0.99	608.1 (5.90)	0.97	8.5 (0.41%)	1.05	194 (0.82)	1.00
600	233.20 (13.6)	1.06	531.6 (7.72)	1.01	610.1 (9.11)	0.97	7.6 (0.16%)	0.94	189 (1.89)	0.97
700	223.90 (7.42)	1.02	433.7 (19.47)	0.83	555.6 (17.86)	0.89	8.5 (0.45%)	1.05	197 (0.82)	1.02
800	207.40 (9.88)	0.94	371.8 (14.52)	0.71	643.0 (6.98)	1.03	8.6 (0.61%)	1.06	207 (2.16)	1.07
900	222.37 (13.95)	1.01	344.0 (13.63)	0.66	604.4 (3.38)	0.97	8.2 (0.38%)	1.01	187 (2.16)	0.96
1000	221.67 (16.19)	1.01	446.4 (13.22)	0.85	629.6 (8.46)	1.01	7.1 (0.86%)	0.88	200 (4.97)	1.03

Note: The values in parentheses represent the standard deviation of the corresponding results

Table 4
CIW test results (mean value)

Temperature (°C)	Elastic Modulus		Yield Strength		Ultimate Strength		Elongation after Fracture		Brinell Hardness	
	$E/(GPa)$	η_E	$f_y/(MPa)$	η_{fy}	$f_u/(MPa)$	η_{fu}	$\varepsilon_T(\%)$	$\eta_{\varepsilon T}$	HB	η_{HB}
20	220.10 (11.63)	1.00	524.4 (11.16)	1.00	625.9 (9.39)	1.00	8.1 (0.06%)	1.00	194 (1.63)	1.00
200	223.80 (15.63)	1.02	526.3 (5.84)	1.00	621.1 (5.06)	0.99	8.1 (0.20%)	1.00	193 (2.94)	0.99
400	216.60 (6.65)	0.98	526.6 (6.32)	1.00	615.4 (5.49)	0.98	7.5 (0.06%)	0.93	190 (3.56)	0.98
500	224.00 (17.46)	1.02	517.8 (12.15)	0.99	609.1 (11.77)	0.97	8.1 (0.14%)	1.00	199 (1.41)	1.03
600	227.87 (0.69)	1.04	524.2 (5.23)	1.00	614.3 (7.21)	0.98	8.0 (0.26%)	0.99	188 (2.05)	0.97
700	223.90 (17.27)	1.02	530.8 (7.03)	1.01	606.2 (7.15)	0.97	7.7 (0.43)	0.95	200 (2.16)	1.03
800	233.80 (11.02)	1.06	428.3 (10.56)	0.82	735.3 (16.73)	1.17	6.6 (0.87%)	0.81	199 (2.94)	1.03
900	211.75 (10.74)	0.96	703.5 (9.68)	1.34	923.7 (23.90)	1.48	4.9 (0.32%)	0.60	230 (6.38)	1.19
1000	228.05 (6.65)	1.04	793.7 (14.71)	1.51	983.4 (18.65)	1.57	4.3 (0.08%)	0.53	250 (4.32)	1.29

Note: The values in parentheses represent the standard deviation of the corresponding results

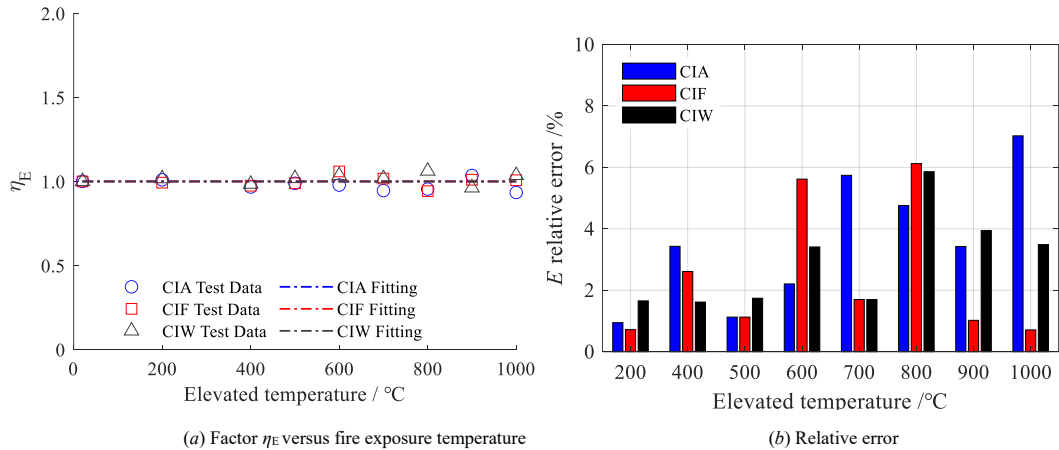


Fig. 8 Fitting results and prediction errors

4.2. Elastic modulus

The variation of the factor η_E with fire exposure temperature and the

corresponding prediction error is illustrated in Fig. 8. The factor η_E exhibits only slight fluctuations under the three cooling methods, with standard deviations of 3.4%, 3.3%, and 3.1% for the CIA, CIF, and CIW conditions, respectively. These

results indicate that the effects of exposure temperature and cooling rate on the elastic modulus of Q460qENH weathering steel are minimal, which is consistent with findings reported in previous studies [6,8,10-13].

As shown in Fig. 8(b), the relative prediction errors of the fitted equations under all three cooling methods are generally within 6.0%, with a maximum value of approximately 7.1% observed for the CIA condition at 1000 °C. The overall trend indicates that the prediction error tends to increase with rising temperature, particularly under the cooling-in-air condition. Nevertheless, the relatively low and stable error magnitudes across all temperature levels and cooling methods further demonstrate the reliability and robustness of the proposed prediction model for η_E .

4.3. Yield strength

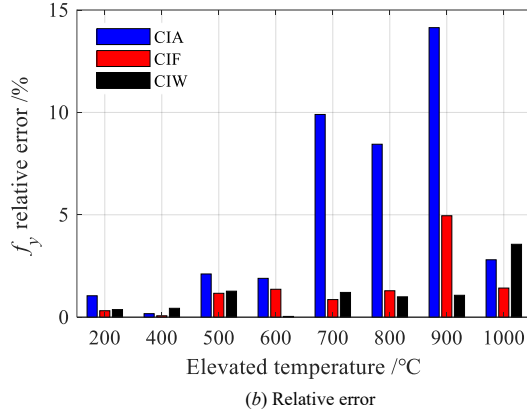
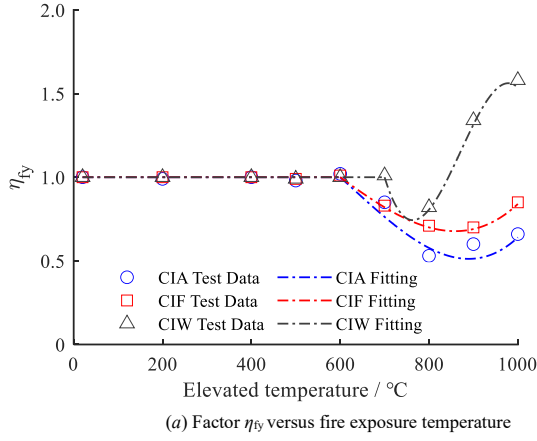


Fig. 9 Fitting results and prediction errors

The temperature transition point for the CIW condition is determined to be 700 °C, as η_{fy} at this exposure temperature equals 1.01, indicating no significant change in post-fire yield strength compared with room-temperature specimens. At 800 °C, the post-fire yield strength of the CIW800 specimen decreases by 18.0% relative to room temperature. However, as the temperature further increases, η_{fy} sharply rises to 1.34 and 1.51 for CIW900 and CIW1000, respectively.

Based on the variation of η_{fy} under the CIA, CIF, and CIW conditions for Q460qENH weathering steel, cubic polynomial fitting was employed to establish predictive equations, as shown in Eqs. (1) - (3). As illustrated in Fig. 9(b), the fitted models exhibit satisfactory accuracy, with most relative prediction errors remaining below 10% and the maximum error not exceeding 14.2%. These results confirm the reliability and robustness of the proposed equations across different high-temperature exposures and cooling methods.

Prediction of variation factor $\eta_{fy, CIA}$ for CIA ($R^2=0.92$):

$$\eta_{fy, CIA} = \frac{f_{y, T}}{f_{y, 20}} = \begin{cases} 1 & 20^\circ\text{C} \leq T \leq 600^\circ\text{C} \\ 2.52\left(\frac{T}{600} - 1\right)^3 - 0.33\left(\frac{T}{600} - 1\right)^2 - 1.44\left(\frac{T}{600} - 1\right) + 1 & 600^\circ\text{C} < T \leq 1000^\circ\text{C} \end{cases} \quad (1)$$

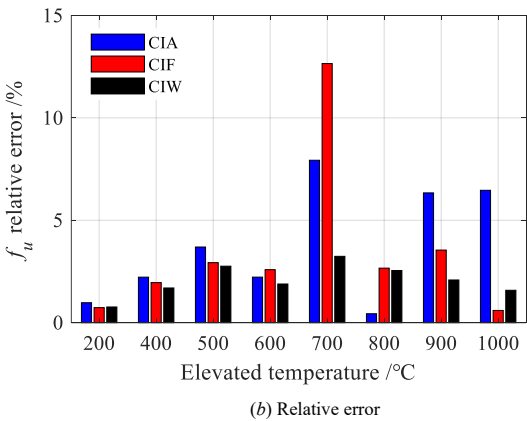
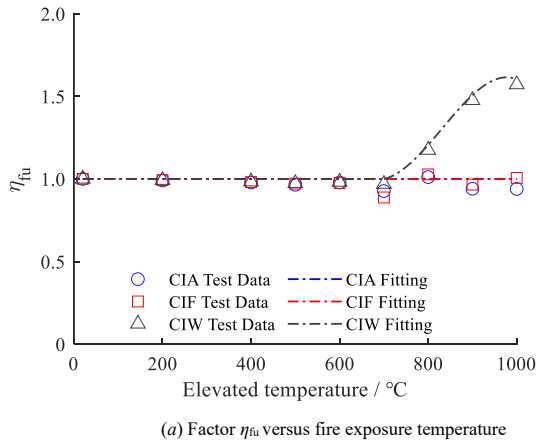


Fig.10 Fitting results and prediction errors

The variation of the factor η_{fy} with fire exposure temperature is illustrated in Fig. 9(a). When the post-fire temperature is below 600 °C, η_{fy} ranges from 0.98 to 1.02 under the CIA condition and from 0.99 to 1.01 under the CIF condition. However, as the exposure temperature increases from 700 °C to 1000 °C, the post-fire yield strength of Q460qENH weathering steel exhibits a decreasing trend followed by a recovery. Consequently, the temperature transition point for mechanical properties can be identified at approximately 600 °C for both CIA and CIF conditions. Specifically, for CIA specimens, η_{fy} reaches its minimum value of 0.53 at 800 °C and then recovers to 0.66 at 1000 °C. Similarly, CIF specimens show a minimum η_{fy} of 0.66 at 800 °C, followed by a rebound to 0.85 at 1000 °C. These results indicate that after high-temperature exposure, the effect of CIF on yield strength is less pronounced than that of CIA, which can be attributed to the slightly higher cooling rate under CIF compared with CIA.

Prediction of variation factor $\eta_{fy, CIF}$ for CIF ($R^2=0.98$):

$$\eta_{fy, CIF} = \frac{f_{y, T}}{f_{y, 20}} = \begin{cases} 1 & 20^\circ\text{C} \leq T \leq 600^\circ\text{C} \\ 1.80\left(\frac{T}{600} - 1\right)^3 + 0.18\left(\frac{T}{600} - 1\right)^2 - 1.16\left(\frac{T}{600} - 1\right) + 1 & 600^\circ\text{C} < T \leq 1000^\circ\text{C} \end{cases} \quad (2)$$

Prediction of variation factor $\eta_{fy, CIW}$ for CIW ($R^2=0.98$):

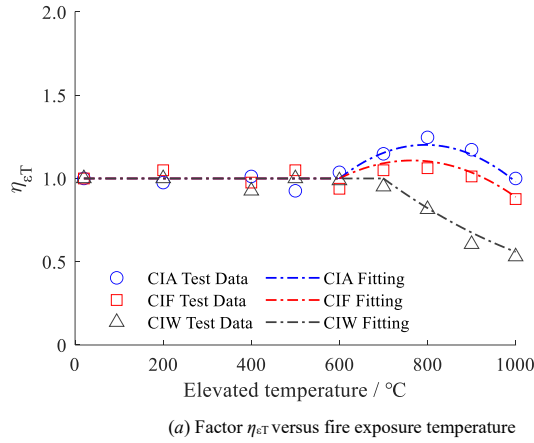
$$\eta_{fy, CIW} = \frac{f_{y, T}}{f_{y, 20}} = \begin{cases} 1 & 20^\circ\text{C} \leq T \leq 700^\circ\text{C} \\ -56.59\left(\frac{T}{700} - 1\right)^3 + 41.65\left(\frac{T}{700} - 1\right)^2 - 6.13\left(\frac{T}{700} - 1\right) + 1 & 700^\circ\text{C} < T \leq 1000^\circ\text{C} \end{cases} \quad (3)$$

4.4. Ultimate strength

The variation of the factor η_{fu} with fire exposure temperature is illustrated in Fig. 10(a). It is evident that under both CIA and CIF conditions, the influence of elevated temperature exposure on the post-fire ultimate strength is negligible.

For the CIW condition, when the exposure temperature remains below the transition point of 700 °C, the post-fire ultimate strength of Q460qENH weathering steel remains largely unchanged. However, once the exposure temperature exceeds this transition point, the post-fire ultimate strength exhibits a pronounced increase, reaching values of 1.17, 1.48, and 1.57 for the CIW800, CIW900, and CIW1000 specimens, respectively. Moreover, the relative prediction errors shown in Fig. 10(b) confirm the satisfactory accuracy of the fitted model for η_{fu} . Across all temperature levels, most error values remain below 10%, with slightly higher deviations observed near the transition temperature region. These results demonstrate that the proposed model effectively captures the temperature-dependent variation in post-fire ultimate strength, particularly under the CIW condition. A mathematical model describing the variation in the post-fire ultimate strength of Q460qENH weathering steel under the CIW condition is presented in Eq. (4).

Prediction of variation factor $\eta_{fu, CIW}$ for CIW ($R^2=0.92$):



$$\eta_{fu, CIW} = \frac{f_{u,T}}{f_{u,20}} = \begin{cases} 1 & 20^\circ\text{C} \leq T \leq 700^\circ\text{C} \\ -17.19\left(\frac{T}{700} - 1\right)^3 + 9.69\left(\frac{T}{700} - 1\right)^2 + 0.41\left(\frac{T}{700} - 1\right) + 1 & 700^\circ\text{C} < T \leq 1000^\circ\text{C} \end{cases} \quad (4)$$

4.5. Elongation after fracture

The ductility of steel after fire exposure can be evaluated by examining the reduction in elongation after fracture. This parameter represents the percentage change in elongation relative to the specimen's original length, serving as a quantitative indicator of its deformation capacity prior to failure. The factor η_{eT} , which characterizes the post-fire elongation after fracture under the three cooling methods, is compared in Fig. 11(a).

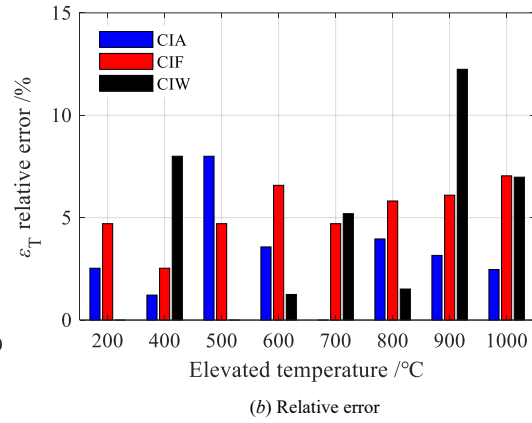


Fig. 11 Fitting results and prediction errors

For all three cooling methods, the effect of elevated temperature exposure on elongation after fracture is negligible below their respective temperature transition points. When the exposure temperature exceeds the transition point of 600 °C for the CIA and CIF specimens, the elongation after fracture initially increases and then decreases. Specifically, it reaches peak values of 1.25 and 1.06 for the CIA and CIF specimens at 800 °C, respectively, and subsequently decreases to approximately 1.00 and 0.88 at 1000 °C. In contrast, for the CIW specimens, once the exposure temperature exceeds the transition point of 700 °C, the factor η_{eT} undergoes a significant decline, reaching a minimum value of 0.53 at an exposure temperature of 1000 °C. The fitted equations for the factor η_{eT} under the different cooling methods are provided in Eqs. (5)–(7).

As shown in Fig. 11(b), the relative prediction errors for η_{eT} vary with temperature and cooling method. For the CIA specimens, the errors remain below 8.0% at all temperatures. The CIF specimens exhibit relatively stable deviations across the entire temperature range, with a maximum error of approximately 7.1%. Under the CIW condition, slightly higher deviations are observed, reaching a peak error of 12.2% at 900 °C. These results indicate that the fitted models capture the temperature-dependent post-fire elongation after fracture behavior with satisfactory accuracy across all cooling scenarios.

Prediction of variation factor $\eta_{eT, CIA}$ for CIA ($R^2=0.96$):

$$\eta_{eT, CIA} = \frac{\varepsilon_{T,T}}{\varepsilon_{T,20}} = \begin{cases} 1 & 20^\circ\text{C} \leq T \leq 600^\circ\text{C} \\ -1.91\left(\frac{T}{600} - 1\right)^2 + 1.24\left(\frac{T}{600} - 1\right) + 1 & 600^\circ\text{C} < T \leq 1000^\circ\text{C} \end{cases} \quad (5)$$

Prediction of variation factor $\eta_{eT, CIF}$ for CIF ($R^2=0.92$):

$$\eta_{eT, CIF} = \frac{\varepsilon_{T,T}}{\varepsilon_{T,20}} = \begin{cases} 1 & 20^\circ\text{C} \leq T \leq 600^\circ\text{C} \\ -1.43\left(\frac{T}{600} - 1\right)^2 + 0.86\left(\frac{T}{600} - 1\right) + 1 & 600^\circ\text{C} < T \leq 1000^\circ\text{C} \end{cases} \quad (6)$$

Prediction of variation factor $\eta_{eT, CIW}$ for CIW ($R^2=0.98$):

$$\eta_{eT, CIW} = \frac{\varepsilon_{T,T}}{\varepsilon_{T,20}} = \begin{cases} 1 & 20^\circ\text{C} \leq T \leq 700^\circ\text{C} \\ 0.76\left(\frac{T}{600} - 1\right)^2 - 1.35\left(\frac{T}{600} - 1\right) + 1 & 700^\circ\text{C} < T \leq 1000^\circ\text{C} \end{cases} \quad (7)$$

4.6. Brinell hardness

Hardness serves as a critical indicator for evaluating the mechanical properties of metallic materials, representing their resistance to localized deformation—particularly plastic deformation, indentation, and scratching—under static loading conditions. Materials with higher hardness typically demonstrate superior wear resistance, and this parameter reflects, to some extent, the overall mechanical performance of the material. A well-established correlation exists between Brinell hardness and other mechanical properties, such as ultimate tensile strength [26]. Consequently, hardness testing provides a valuable, non-destructive means of indirectly assessing material strength, particularly in scenarios where destructive testing is impractical. However, studies focusing on the post-fire hardness characteristics of structural steels remain relatively limited.

The pronounced increase in strength and Brinell hardness, along with the significant reduction in elongation observed in the CIW800 - 1000 specimens, suggests a notable microstructural transition induced by rapid water cooling from elevated temperatures. When high-strength steels are cooled rapidly from the austenitic range, martensitic transformation tends to occur, typically characterized by a high dislocation density, refined lath structures, and elevated internal strain energy—features that contribute to increased mechanical strength and hardness [27]. In this study, the CIW specimens subjected to cooling from 800 °C and above exhibited more than a 20% increase in hardness, accompanied by marked changes in fracture morphology - such as reduced dimple size, smoother fracture surfaces, and lower ductility indices - as shown in Figs. 6(h), 6(i), and 11(a). These trends are consistent with the characteristics of martensitic transformation reported in previous studies on fire-exposed steels undergoing rapid quenching [20,21], indicating that phase transformation likely governs the observed mechanical behavior. Fig. 12 illustrates the variation of the η_{HB} factor for Q460qENH weathering steel as a function of fire exposure temperature and prediction error.

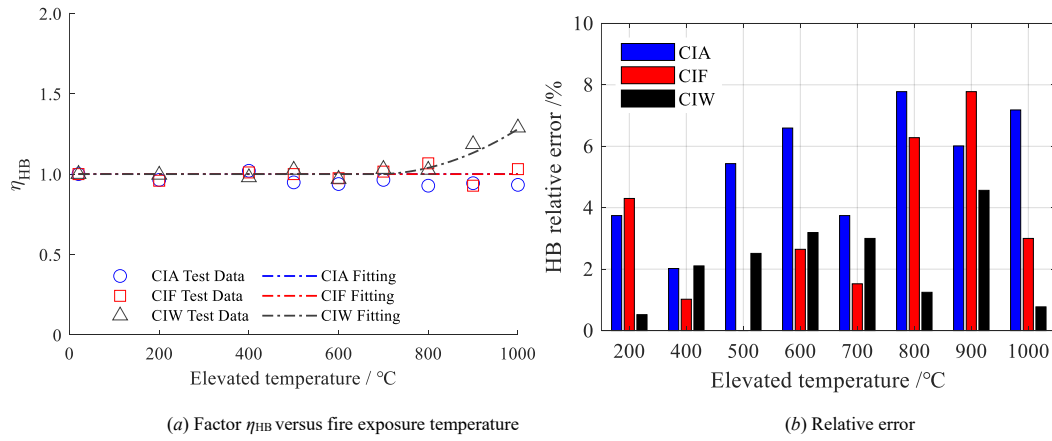


Fig. 12 Fitting results and prediction errors

The results demonstrate that under both CIA and CIF conditions, elevated temperature exposure has minimal influence on post-fire Brinell hardness. In contrast, under the CIW condition, a significant increase in Brinell hardness is observed once the exposure temperature exceeds the transition point of 700 °C. Specifically, compared with the room-temperature values, the Brinell hardness increases by 2.6%, 18.6%, and 28.9% at exposure temperatures of 800 °C, 900 °C, and 1000 °C, respectively. This phenomenon can be attributed to the accelerated cooling rate above 700 °C, which promotes martensitic transformation in the weathering steel and leads to a substantial enhancement in hardness during water cooling [27]. As shown in Fig. 12(b), the relative prediction errors for η_{HB} vary with both temperature and cooling method. Under the CIA and CIF conditions, the errors range from 1.0% to 7.8%, with the largest deviations occurring at 800 °C and 900 °C, respectively. In contrast, the CIW condition exhibits lower overall errors, mostly below 4.6%, reaching as low as 0.8% at 1000 °C. These results indicate that the prediction model performs well across all conditions, exhibiting particularly stable accuracy under the CIW regime. Based on these observations, a mathematical model describing the variation of Brinell hardness for Q460qENH weathering steel under the CIW condition is proposed in Eq. (8).

Prediction of variation factor $\eta_{HB, CIW}$ for CIW ($R^2=0.95$):

$$\eta_{HB, CIW} = \frac{HB_{st}}{HB_{20}} = \begin{cases} 1 & 20^\circ\text{C} \leq T \leq 700^\circ\text{C} \\ 1.33\left(\frac{T}{700} - 1\right)^2 + 0.08\left(\frac{T}{700} - 1\right) + 1 & 700^\circ\text{C} < T \leq 1000^\circ\text{C} \end{cases} \quad (8)$$

These predictive equations, established based on statistically averaged results from three specimens per condition, demonstrate good accuracy, with relative errors mostly within 15.0% across all temperature levels and cooling regimes, as illustrated in Figs. 8-12. Their robustness and low prediction deviations confirm their reliability for practical applications. Moreover, the equations cover a wide range of fire exposure temperatures (200 °C to 1000 °C) and account for three representative cooling conditions - cooling in air, cooling in foam, and cooling in water - thus providing a flexible and versatile toolset for post-fire performance evaluation.

5. Comparisons and discussion

This section presents a comparative analysis of the mechanical properties of Q460qENH weathering steel and other steel grades after fire exposure. Table 5 summarizes the steel grades and corresponding test conditions reported in the literature, focusing on the comparison of key mechanical property parameters, including elastic modulus, yield strength, and ultimate strength.

Table 5
Comparison of mechanical properties of different steel grades

Steel Grade	Source	Heating Temperature (°C)	Cooling method	Cross section shape
Q235	Lu [28]	100~1000	CIA, CIW	Rectangle
Q345	Lu [28]	100~1000	CIA, CIW	Rectangle
Q345	Zhang [13]	200~900	CIF	Rectangle
Q355	Zhang [11]	200~900	CIF, CIW	Rectangle
Q355qNH	Zhang [5]	300~1000	CIA, CIW	Rectangle
Q420	Lu [28]	100~1000	CIA, CIW	Rectangle
Q460	Zhang [19]	200~900	CIF	Rectangle
Q460	Wang [29]	300~900	CIA, CIW	Rectangle
Q550	Song [30]	200~900	CIA, CIW	Rectangle
Q620	Zhou [12]	300~900	CIA, CIF, CIW	Rectangle
Q690	Zhang [19]	200~900	CIF	Rectangle
Q690	Li [31]	300~900	CIA, CIW	Rectangle
Q890	Zeng [32]	200~1000	CIA, CIW	Rectangle
Q960	Wang [33]	300~900	CIA, CIW	Circular
S460	Qiang [34]	300~1000	CIA	Rectangle
S690	Qiang [34]	100~1000	CIA	Rectangle
S960	Qiang [8]	300~1000	CIA	Rectangle
A36	Sajid [27]	500~1000	CIA, CIW	Hollow Circular
A572	Sajid [35]	500~1000	CIA, CIW	Hollow Circular
A992	LEE J [36]	200~1000	CIA, CIW	Rectangle

Fig. 13 illustrates the variation in the elastic modulus of various steels with increasing fire-exposure temperature under different cooling conditions. Under the CIA condition, for Q960, S460, S690, and S960 steels, when the fire-exposure temperature exceeds 600 °C, the elastic modulus decreases significantly, indicating a notable degradation in the mechanical properties of these steels under high-temperature conditions. However, for the remaining steels, the elastic modulus exhibits only minor fluctuations, with a variation range of -14.0% to 7.0%. Similarly, under the CIF condition, the elastic modulus

of Q460qENH and the other steels remains stable with increasing fire-exposure temperature, exhibiting only slight variations across the entire temperature range. Under the CIW condition, as the fire-exposure temperature increases, the elastic modulus of each steel generally shows a slight downward trend, though the overall variation range remains small. Notably, at a fire-exposure temperature of 1000 °C, the CIA1000 specimen exhibits an η_E of 0.93, while the CIW1000 specimen shows an η_E of 1.04, both indicating good stability of the elastic modulus after high-temperature cooling.

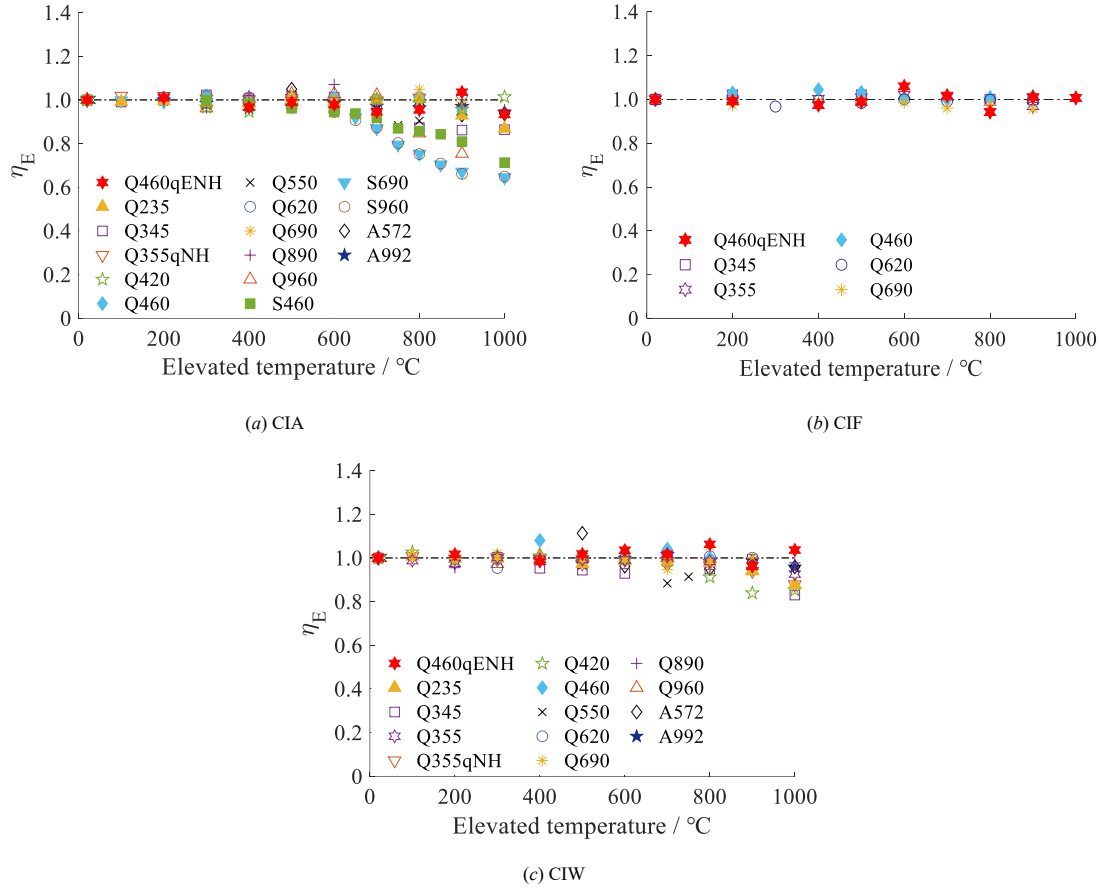


Fig. 13 Comparison on post-fire elastic modulus

Fig. 14 presents a comparison of the post-fire yield strength of different steel grades. When the fire-exposure temperature exceeds 800 °C, both Q460qENH and Q960 steels exhibit strength recovery, with their η_{fy} increasing by 13% and 16%, respectively. Except for Q235 and Q345 steels, the yield strength of other steels increases following water cooling at temperatures above 800 °C. Specifically, after exposure to a high temperature of 900 °C, the yield strengths of A36 and A572 steels under CIW conditions are 143% and 103% higher, respectively, than those under CIA conditions. Microstructural analysis indicates that this phenomenon is attributed to the transformation of high-temperature austenite into ferrite and the strengthening phase, martensite, resulting from water cooling [37,38]. According to the evaluation code T/CECS 252:2019 [39], under the CIA condition, the yield strength of Q460qENH weathering steel meets the code requirements when the fire-exposure temperature is below 600 °C. However, when the temperature exceeds 600 °C, the yield strengths of Q550, Q620, Q690, Q890, Q960, S690, S960, and A572 steels decrease significantly, with reductions exceeding the allowable limits specified by the code. Under the CIF condition, the post-fire yield strength of most steels shows a trend consistent with that observed under CIA conditions. Notably, high-strength steels such as Q460, Q620, and Q690 exhibit moderate reductions in yield strength when the fire-exposure temperature exceeds 700 °C. These results suggest that although foam cooling provides faster initial heat extraction, its subsequent insulating effect slows the overall cooling process, resulting in thermal histories that closely resemble those of natural air cooling. Consequently, the mechanical degradation patterns under CIF and CIA conditions tend to

converge.

Fig. 15 presents a comparison of the post-fire ultimate strength of different steel grades. The results indicate that under CIA conditions, the post-fire strength of Q460qENH weathering steel and A992 steel, after exposure to various high temperatures, still satisfies the strength requirements specified in GB/T 19879-2023 [40]. For the other steels, once the fire-exposure temperature exceeds the critical threshold, the ultimate strength falls below the lower limit defined by the standard. Under CIF conditions, with the exception of Q460qENH weathering steel, all other tested steel grades exhibit varying degrees of post-fire degradation in ultimate strength when the exposure temperature exceeds 700 °C. Notably, for Q460, Q620, and Q690 steels, the ultimate strength decreases by approximately 30% after exposure to 900 °C. This degradation trend under CIF conditions closely resembles that observed under CIA. However, the slightly improved strength retention observed under CIF suggests that foam cooling may provide a modest thermal-buffering effect, mitigating the extent of high-temperature damage compared with natural air cooling. Under CIW conditions, the ultimate strength of each steel generally follows the same trend as its yield strength. When the temperature exceeds 700 °C, the ultimate strength of A36 and A572 steels increases significantly, with the factor η_{fu} rising by 154% and 102%, respectively, after exposure to 900 °C. Notably, under CIW conditions, the ultimate strength of Q460qENH weathering steel increases by 60% following exposure to 1000 °C, indicating exceptional post-fire strengthening behavior under rapid water quenching.

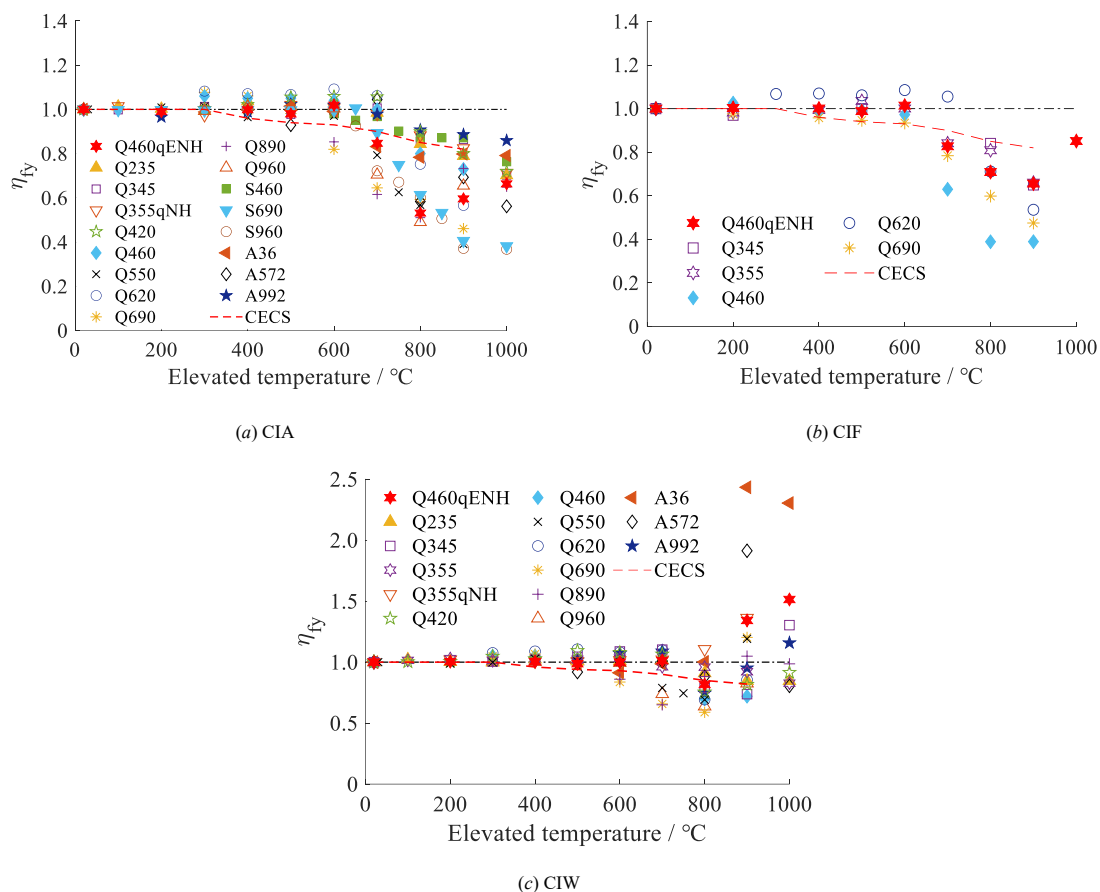


Fig. 14 Comparison on post-fire yield strength

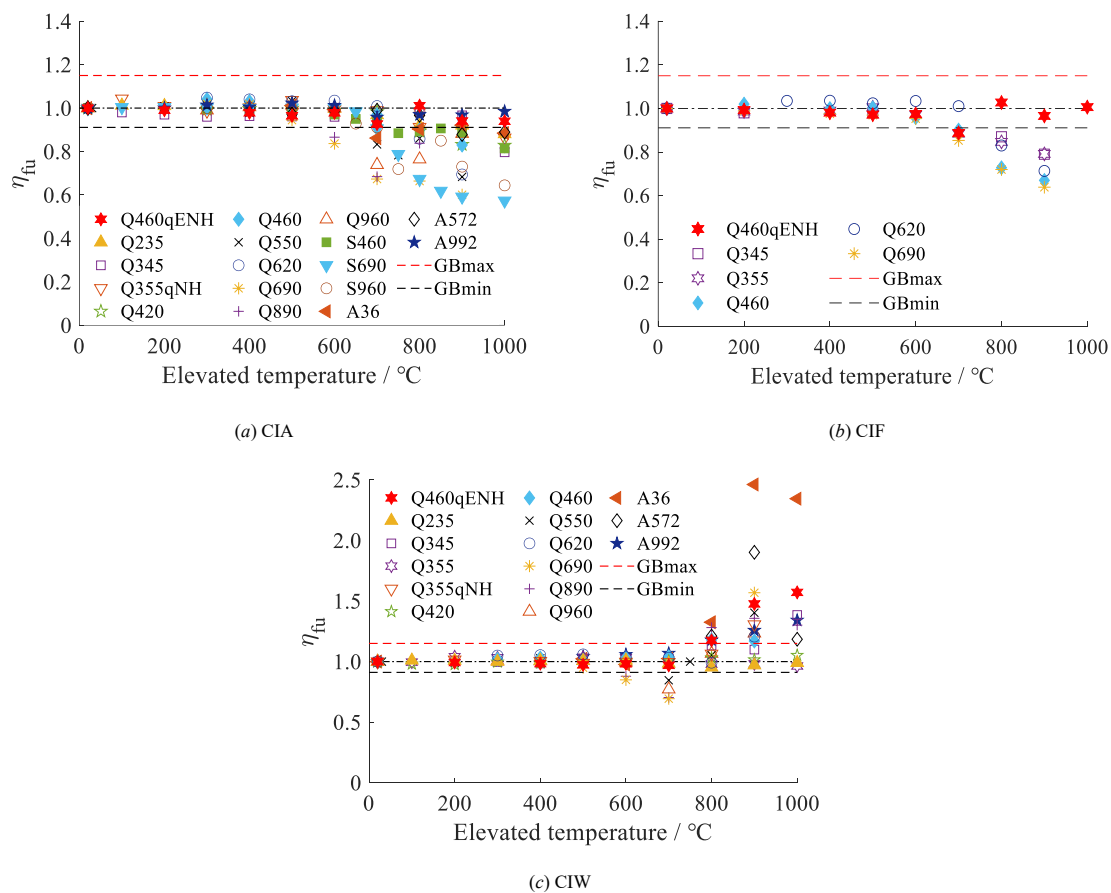


Fig. 15 Comparison on post-fire ultimate strength

Based on the comparative analysis of the experimental data, Q460qENH weathering steel exhibits outstanding post-fire mechanical performance under

both CIA and CIW conditions. These results establish a solid foundation for evaluating the residual mechanical properties of weathering steel after fire

exposure. Furthermore, the proposed predictive models, which correlate temperature with key mechanical parameters, not only facilitate the quantitative assessment of performance degradation but also provide valuable theoretical support for the development and refinement of fire-resistant design standards for steel structures.

6. Conclusions

(1) After exposure to elevated temperatures, Q460qENH weathering steel exhibits notable changes in both surface morphology and microstructure, depending on the applied cooling method. The resulting surface characteristics serve as indicators of the thermal history and cooling conditions experienced by the steel. Microstructural observations suggest that water immersion cooling (CIW) leads to a pronounced reduction in material toughness, underscoring the sensitivity of Q460qENH to rapid quenching effects.

(2) The phase transformation temperature for the CIA and CIF specimens is identified as 600 °C. Below this threshold, all mechanical properties remain stable. However, when the temperature exceeds 600 °C, the elastic modulus, ultimate tensile strength, and Brinell hardness exhibit only minor variations, whereas the yield strength and elongation after fracture change significantly. For the CIW specimens, the phase transformation temperature is determined to be 700 °C, above which all mechanical properties undergo pronounced changes.

(3) A comparative investigation was conducted between Q460qENH weathering steel and other types of steel, categorized as low-carbon steel (e.g., Q235, A36), mild steel (e.g., Q345, Q355, Q355qNH), and high-strength steel (e.g., Q420, Q460, Q550, Q620, Q690, Q890, Q960, S460, S690, S960, A572, A992), to evaluate the post-fire mechanical performance of Q460qENH. The results indicate that Q460qENH exhibits excellent fire resistance.

Availability of data and materials

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

Disclosure Statement

No potential conflict of interest was reported by the author(s).

Funding

This work was supported by Major Scientific & Technological Innovation Projects of Shandong Province, P. R. China (No. 2021CXGC011204).

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THEORETICAL FEASIBILITY STUDY OF THE NEW STEEL GRATING WEB COMPOSITE BEAM

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ABSTRACT

To reduce the self-weight and the steel consumption of the traditional steel-concrete composite beam, a new steel grating web composite (SGWC) beam is proposed by introducing a steel grating instead of the solid steel beam web. The uniaxial behavior, mechanical performance, load-displacement curve, and failure mechanism of the SGWC beam were studied, and the influence of steel grating size on their bearing capacity was thoroughly investigated. The results indicate that a 21.5% weight reduction in the SGWC beam versus traditional steel beams, and the strength, stiffness, and stability meet the requirements of the Chinese specification. In the elastoplastic analysis, the failure process of the SGWC beam can be divided into the elastic stage, the elastoplastic stage, and the plastic stage. The SGWC beam's load-bearing capacity reduces as grating spacing grows, but improves with greater grating thickness and width.

ARTICLE HISTORY

Received: 1 April 2025
Revised: 4 September 2025
Accepted: 20 September 2025

KEYWORDS

Composite beam;
Steel grating web;
Feasibility study;
Load-displacement curve;
Failure mechanism

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1. Introduction

Bridges play a vital role in transportation as an infrastructure that crosses valleys and rivers and connects the transport between the two places [1, 2]. The steel-concrete composite beam comprises a concrete slab and a steel beam, which are connected by connectors and are stressed together. When used as a flexural member, these composite beams optimally combine concrete's compressive strength with steel's tensile capacity while avoiding the cracking and stability issues of pure RC or steel beams. Therefore, steel-concrete composite beams have become prevalent in bridge applications [3-5].

In recent years, as the span of bridge structures has increased, the dead weight and steel consumption of traditional steel-concrete composite I-beams have significantly risen. This not only results in higher material costs but also imposes more stringent requirements on the substructure and foundation. To address this challenge, scholars have proposed various technical solutions to enhance the bearing capacity of steel-concrete composite beams while reducing the structural weight. These solutions focus on enhancing concrete slabs, connectors, and steel webs.

Enhancing the performance of concrete slabs can significantly improve the application of steel-concrete composite beams. Men et al. [6] investigated the mechanical behavior of composite beams with UHPC-normal concrete composite slabs under negative bending moments and found that their application to composite beams can greatly enhance crack resistance while preserving bending capacity. Zha et al. [7] added UHPC to the traditional concrete bridge deck to form a composite bridge deck, and carried out several sets of tests to compare the bending properties of the composite beams. The results demonstrate the novel composite deck enhances beam performance by 13%. Jiang et al. [8] developed a prefabricated UHPC-steel pipe composite beam. The test shows that the composite beam has excellent bending performance and material utilization. Guo et al. [9] enhanced the structural bearing capacity and durability of steel-concrete composite beams by incorporating fiber-reinforced polymer reinforcement into the concrete slab.

As a critical component linking steel beams and concrete slabs, the arrangement of connectors significantly influences the bearing capacity of composite beams. Ding et al. [10] presents an angular connector that enhances the stiffness and anti-slip characteristics of composite beams. Luo et al. [11] developed innovative demountable bolted shear connectors, enhancing prefabricated composite beam performance through roll-out testing. Tzouka et al. [12] established a 3D numerical model to analyze novel shear connector behavior. This connector allows the bridge to be dismantled and can help to replace the aging bridge components, and the experimental results verify the reliability of the proposed model. Yang et al. [13] introduced a steel-concrete composite beam without a top steel flange and examined the application of various shear connectors within this structure. The results demonstrate that all connectors ensure the overall performance of the structure.

In traditional I-beams, the typical bending failure mode involves the

simultaneous crushing of the concrete bridge deck and yielding of the bottom steel beam flange [14]. Structural efficiency is constrained by the weight of the solid steel beam, while the web's contribution to the structure's bearing capacity is also limited. Therefore, enhancing the web design is crucial for optimizing composite beam performance. The corrugated steel web is an effective design choice. To improve the cracking problem of composite beams, Pei et al. [15] replaced the flat steel web with corrugated steel webs, and carried out model tests to study the effect of web form on the bending and crack resistance of composite I-beams. Test results demonstrate the corrugated web design's superior crack resistance in bridge applications. Wu et al. [16] further demonstrated that corrugated steel webs are more advantageous in composite beams with spans exceeding 30 m. Honeycomb beam structures are widely used in civil construction in European and American countries due to their cost-effectiveness and efficiency [17]. Zhao et al. [18] compared the static loading test of the partially clad steel-concrete composite structure and the partially covered honeycomb steel-concrete composite beam to investigate the influence of the beam by the opening of the main steel web. In the case of similar materials, the honeycomb main steel web can improve the beam's bearing capacity compared with the traditional solid steel web by increasing the cross-sectional height through the appropriate opening. Still, it also reduces the material deformation capacity and is more affected by shear force.

Nowadays, with the development of bridge construction in mountainous areas, the steel-concrete composite beams with low requirements for hoisting equipment compared with other beams are a vital direction for bridge designers, and a key research challenge is to improve the engineering economy effectively of these composite beams [19]. As a material with a broad market prospect, steel grating plate has the advantages of high bearing performance and outstanding slip resistance under the same consumables and have been successfully applied to practical engineering [20-25]. Introducing this structure can significantly reduce the large amounts of steel used in I-beams.

Based on the above understanding, this study introduces a novel steel grating web composite (SGWC) beam that replaces the solid steel beam web with a steel grating plate based on the existing steel-concrete composite beam theory and design specifications. Taking the 60m simply-supported SGWC beam as the research object, the reasonable structure size, mechanical performance, and failure mechanism were studied. This research advances sustainable construction through improved composite beams structural performance suitable for prefabricated bridge applications.

2. The proposal of the SGWC beam

Steel gratings play an important role in areas such as tunnel supports, pedestrian bridges, and road paving, and are mostly arranged horizontally to withstand surface loads. Traditional steel-concrete composite beams often fail due to shear along the web at an oblique angle of 45°. Furthermore, referring to steel truss belly composite beam design, the optimal inclination angles for

trusses with and without vertical rods are approximately 50° and 60° , respectively. Therefore, in this study, steel grating is oriented at a 45° angle to effectively resist shear forces, as illustrated in Fig. 1. The load-bearing mechanism is as follows: the oblique steel grating coincides with the main stress direction of the web and bears tensile stress and compressive stress, respectively, to provide strong shear bearing capacity for the composite beam by using the structure characteristics of the steel grating. Vertically arranged steel rods can transmit the vertical forces from the bridge deck and ensure stability.

Based on the above theory, this paper replaces the solid steel beam web with a steel grating for the composite beam. The steel beam of different webs is shown in Fig. 2.

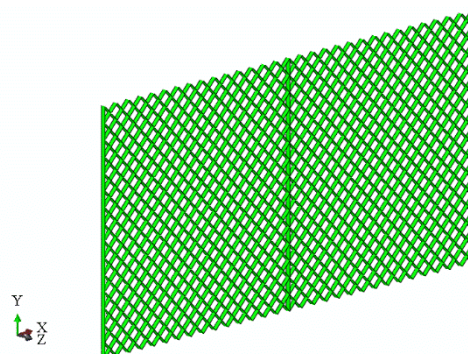
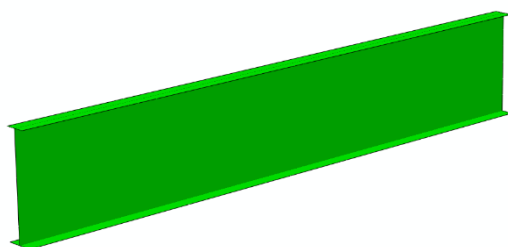
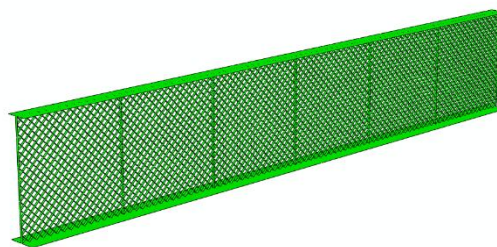


Fig. 1 Schematic diagram of the steel grating web

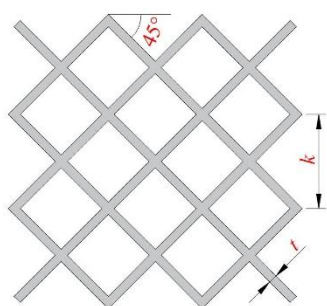


(a) The solid steel beam

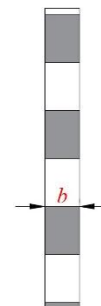


(b) The steel grating steel beam

Fig. 2 Schematic diagram of different web steel beams



(a) Front view



(b) Side view

Fig. 3 Schematic diagram of the steel grating

Fig. 3 illustrates the steel grating schematic, where t is the steel grating thickness, k is the steel grating spacing, the angle between the steel grating and the flange steel plates of the steel beam is 45° , and b is the steel grating width.

For the steel grating web, its sizes should meet the width-to-thickness ratio and slenderness ratio requirements in the Chinese standard [26]. Referring to the Chinese specification [27], the common sizes of the steel grating are modified, and the specific size range is proposed in Table 1.

Table 1
Size parameters of the steel grating (mm)

t	b	k
5-35	30-120	Meets slenderness ratio and b/t limits

Table 2
The sizes of each composite beam (unit: mm)

	Traditional composite beams		The SGWC beam	
Span l	60000			
Beam height	3300			
Concrete slab	Width: 4250; Thickness: 250			
Upper flange steel plate	Width: 600; Thickness: 30			
Lower flange steel plate	Width: 800; Thickness: 45			
Web	Height: 2925	Thickness: 30	Height: 2925	$t=30$, $b=50$, $k=200$

According to the general design diagram of the 60m composite beam, the

solid steel beam web is replaced by the steel grating and the other parameters are unchanged. The structure comprises concrete slabs, flange steel plates, and steel grating. According to the size parameter range of the steel grating proposed above, for the 60m SGWC beam, the specific sizes of each component are presented in Table 2, and the sizes of the traditional composite beams are also given.

Table 3 shows the comparison of the steel consumption of different composite beams with 60m spans. It can be seen from the table that the SGWC beam reduces web weight by 34.72% versus solid webs and cuts total steel weight by 21.5% compared to traditional beams, offering clear economic advantages.

Table 3
Comparison of the steel consumption in the two composite beams

Type	Traditional composite beams/ m^3	The SGWC beam/ m^3	Proportion of steel used /%
Web	5.27	3.44	65.28
Steel beam	8.51	6.68	78.50

Note: The proportion of steel used is the ratio of the steel consumption of the SGWC beam to the steel consumption of the traditional composite beam

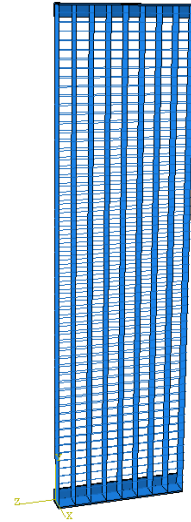
In summary, a composite beam structure using steel grating instead of the solid steel beam web is proposed in this paper. Its overall load-bearing mechanism is as follows: the concrete slab and the upper flange steel plate bear the compressive stress, the lower flange steel plate bears the tensile stress, the oblique steel grating bears the shear stress, and the vertical steel grating is used to transmit the vertical force of the upper flange steel plate. When the SGWC beam is subjected to vertical load, the load is transferred from the concrete slab to the upper flange steel plate, and then to the lower flange steel plate through

the steel grating web. In contrast to the shear stress generated by the web of traditional composite beams, the stress in the steel grating web of the SGWC beam is primarily dominated by axial force.

Compared with the existing composite beams [15–18], the SGWC beam can maximize the shear force-bearing capacity of the web, and it is convenient for replacing local components. Additionally, its unique mesh structure facilitates the dissipation of impact loads, reducing bridge vibration and enhancing driving stability. These findings demonstrate that SGWC beams offer strong stress performance while reducing the structure's weight and steel consumption, making them highly applicable.



(a) Experimental model



(b) Finite element model

Fig. 4 Schematic diagram of steel grating

In the literature experiment, the vertical load is applied by using a jack at the top of the steel grating, and the displacement of the steel grating is controlled by arranging fasteners at the bottom. The analysis utilizes ABAQUS with S4R elements modeling flat steel and C3D8R elements discretizing frame members. The flat steel and the frame are bound constraints, and a common node is set between the flat steel and the steel bar to simulate the constraint effect. Vertical

loads are applied through a reference point coupled to the member's top surface. The loading mode of displacement control is adopted, and the bottom of the component adopts the constraint mode of consolidation. The displacement nephograms in different directions of the steel grating members when they fail are shown in Fig. 5.

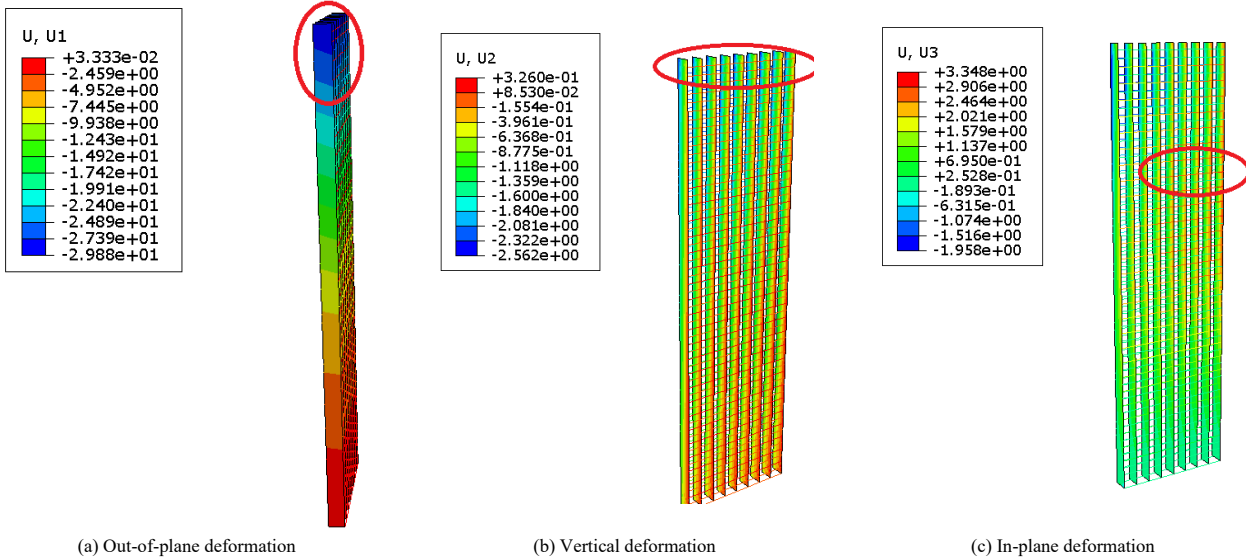


Fig. 5 Deformation nephograms of steel grating (unit: mm)

Table 4
Comparison of uniaxial behavior

Type	Bearing Capacity (kN)	Maximum displacement out of plane (mm)	Maximum displacement in plane (mm)	Maximum vertical displacement (mm)
Experiment	1435.8	-32.99	2.34	2.39
FEA	1369.4	-29.88	3.34	2.56

Table 4 compares experimental displacements from literature with present simulation results. It can be seen from the table that, except that the

displacement of the steel grating member outside the plane is quite different from the calculation results in this paper, close agreement between simulation and experimental results validates the numerical model's accuracy.

Fig. 6 compares experimental and simulated load-displacement curve. As can be seen from the figure, the load displacement curve simulated by finite element analysis in this paper is approximately in a line rising state, indicating that the steel grating has a good linear elastic stage under uniaxial behavior. Moreover, the load displacement curve is very close to that obtained by the experiment, which once again verifies the accuracy of the work in this paper.

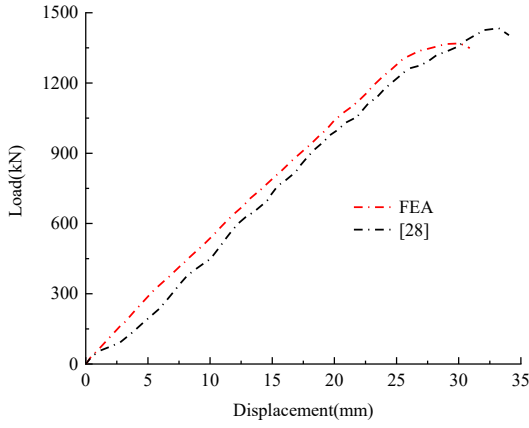


Fig. 6 Load displacement curve of steel grating under uniaxial behavior

4. Finite element analysis of the SGWC beam

In the finite element analysis, the steel is made of Q345, and the concrete slab is made of C50. The concrete slab is modeled with solid elements, and the flange steel plates and steel grating is modeled with shell elements. The concrete slab, flange plate, and steel grating are rigidly connected, which effectively avoids the difficulty of finite element analysis convergence and reduces the time of nonlinear iteration. When beam end constraints are applied to a finite element model, the fixed hinge bearing constrains the x , y and z axis, and the sliding hinge bearing constrains the displacement x -axis and y -axis.

Fig. 7 shows the finite element model of the 60 m SGWC beam. The entire beam is divided into 52,972 elements, with 46,131 nodes in total. There are four element sizes for the concrete slabs, with sizes in the x , y , and z directions as follows: $1 \times 0.1825 \times 0.125 \text{ m}^3$, $1 \times 0.12 \times 0.125 \text{ m}^3$, $0.2 \times 0.1825 \times 0.125 \text{ m}^3$, and $0.2 \times 0.12 \times 0.125 \text{ m}^3$. The upper flange steel plate consists of two element sizes: $0.1 \times 0.0675 \text{ m}^2$ and $0.1 \times 0.03 \text{ m}^2$ in the x and y directions, respectively. The lower flange steel plates come in two sizes: $0.1 \times 0.0925 \text{ m}^2$ and $0.1 \times 0.03 \text{ m}^2$. The steel grating has an element size of $0.1414 \times 0.03 \text{ m}^2$.

The elastic modulus of steel and concrete is 206 GPa and 34.5 GPa, shear modulus is 79.23 GPa and 14.38 GPa, the mass density is $7.85 \times 10^{-5} \text{ N/mm}^3$ and $2.5 \times 10^{-5} \text{ N/mm}^3$, the linear expansion coefficient is $12 \times 10^{-6}/^\circ\text{C}$ and $10 \times 10^{-6}/^\circ\text{C}$, and the Poisson's ratio is 0.3 and 0.2, respectively.

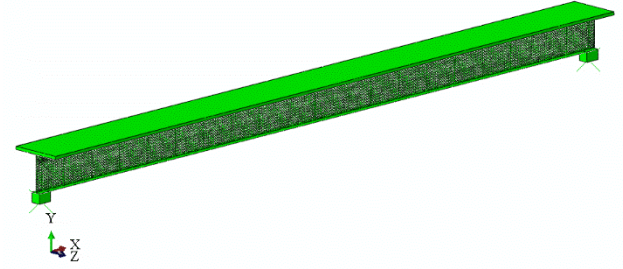


Fig. 7 Schematic diagram of the finite element model of the SGWC beam

4.1. Linear elastic analysis of the SGWC beam

In the strength checking of the SGWC beam, the basic combination of the ultimate limit states is used for loading according to the Chinese specification [29], and the partial safety factor for load is shown in Table 5.

Table 5
Partial safety factor for load

Self-weight	Second-phase load	Lane loads	Temperature	Contraction creep	Wind load
1.2	1.2	1.4	1.05	1.0	1.1

The structural self-weight is automatically computed by the analysis software, and the temperature rise and drop are added through the temperature field in the finite element analysis. The second-phase load is 23.0 kN/m^3 according to the weight of the asphalt concrete, and the thickness is 100 mm. In order to avoid the phenomenon of stress concentration, the 2 m concrete slab in the mid-span of the SGWC beam is coupled, and the concentrated load in the lane load is applied to the 2 m concrete slab. Fig. 8 presents the ultimate limit state longitudinal stress distribution across all beam components. Since the longitudinal stresses are symmetrically distributed along the mid-span, the left half of the beam was selected for the study, and the dimensions of the beam along the axis were reduced by half.

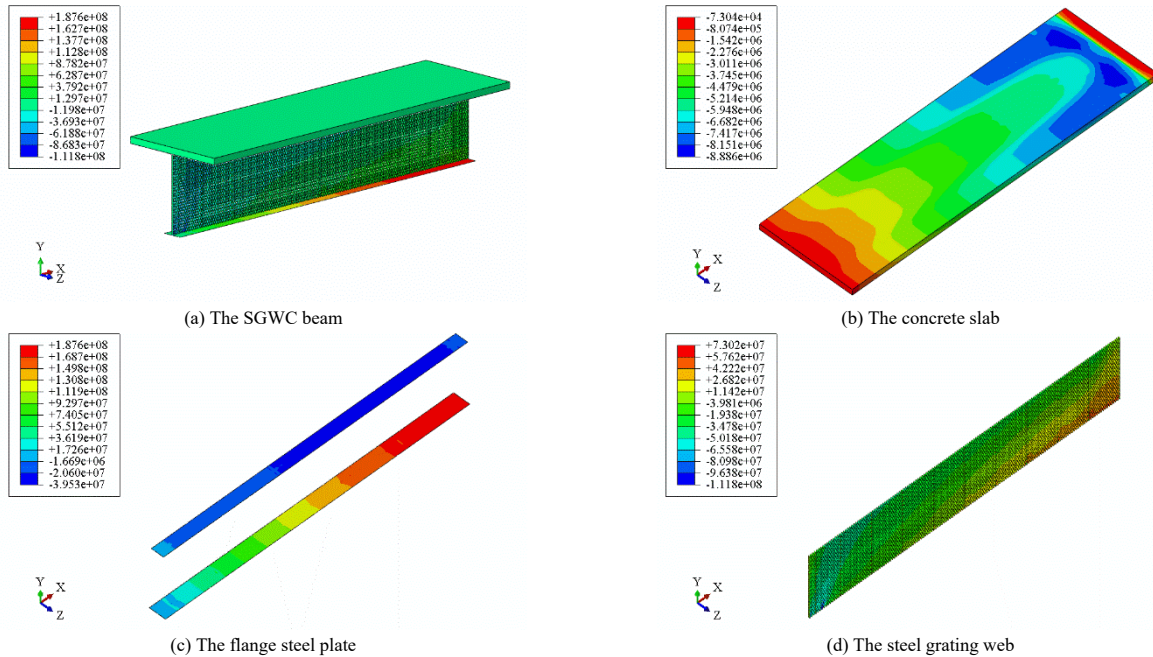


Fig. 8 Stress nephogram of the SGWC beam (unit: Pa)

Fig. 8 shows that the longitudinal stress of the concrete slab is gradually increasing from the bearing to the mid-span. Due to the influence of the way the concentrated load is applied, the maximum compressive stress occurs near the mid-span and its value is 8.89 MPa. Consistent with the concrete slab, flange plate stresses grow gradually toward mid-span while the upper flange remains in compression. The mid-span vicinity reaches maximum compressive stress of 39.53 MPa. The maximum value of the lower flange steel plate is 187.6 MPa in the middle section. The steel grating develops tensile stresses near mid-span

while remaining regions sustain compression. The peak tensile stress occurs in the mid-span section, and its value is 73.02 MPa. Due to the arrangement of bearing at the end of the beam, this position exhibits the steel grating's highest compressive stress: 111.8 MPa. The stress of each component of the SGWC beam is less than the ultimate strength of the material, which meets the requirements of the Chinese specification.

When the stiffness of the SGWC beam is checked, 1.45 is selected as the long-term deflection growth coefficient according to the Chinese specification

[29]. The deflection is calculated according to the frequent combinations in the serviceability limit states. The calculation result is shown in Fig. 9. The figure shows that the maximum deflection of the SGWC beam is 47.2 mm. After considering the long-term increase coefficient, the deflection is 68.44 mm, which is smaller than the deflection limit $l/550$ (109 mm) specified in the Chinese specification [30], indicating that the stiffness of the SGWC beam meets the specification requirements.

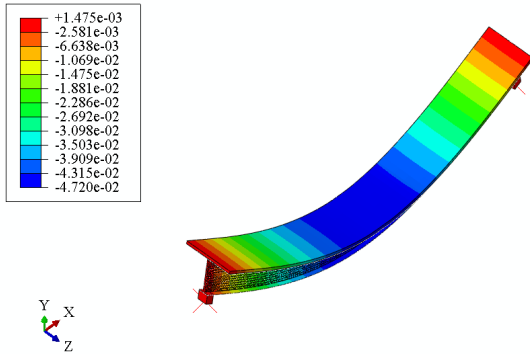


Fig. 9 The deflection of the SGWC beam (unit: m)

In the process of checking the stability of the SGWC beam, the stability coefficient of the first-order eigenvalue with practical significance can theoretically predict the structure's instability shape and ultimate load-bearing capacity. The second buckling mode is used in this paper. The instability shape and eigenvalue stability coefficient of the SGWC beam under the dead load and I-level lane load are calculated, as shown in Fig. 10. The figure shows that when the SGWC beam is destroyed, the first-order eigenvalue coefficient is 5.3438, while the stability safety factor is generally stipulated in the project is not less than 4. This indicates that the stability coefficient of the SGWC beam in the ideal linear elastic state not only meets the required standards but also exceeds the design value by 33.96%. This provides a sufficient safety margin and optimization potential for the stability coefficient of the SGWC beam, even after accounting for initial defects.

According to the results of the strength checking in the ultimate limit states, the stiffness checking in the serviceability limit states, and the stability checking, the structure of the SGWC beam is feasible and meets the specification requirements.

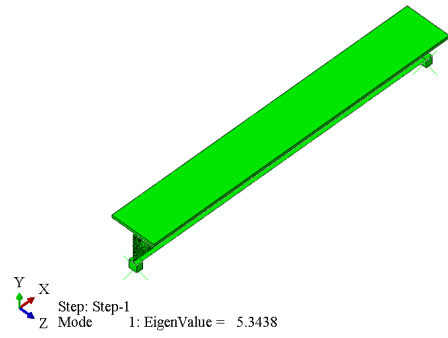


Fig. 10 Eigenvalue buckling mode of the SGWC beam

4.2. Elastoplastic analysis of the SGWC beam

A nonlinear finite element model accounting for material and geometric nonlinearity was developed for the SGWC beam. The constitutive relationship of the steel is given according to the Chinese specification [31], and the uniaxial stress-strain relationship is shown in Fig. 11. In the figure, f_y is the representative value of the yield strength of the steel, f_u is the representative value of the ultimate strength of the steel, ϵ_y is the yield strain of the steel, and ϵ_u is the peak strain of the steel. In this paper, the yield strength of the steel is 345 MPa, the tensile strength is 571 MPa, the elongation is 32%, and the cross-section shrinkage is 60%.

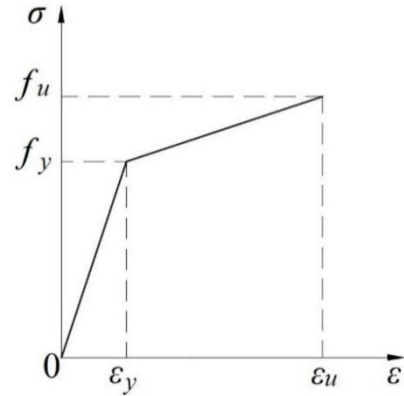
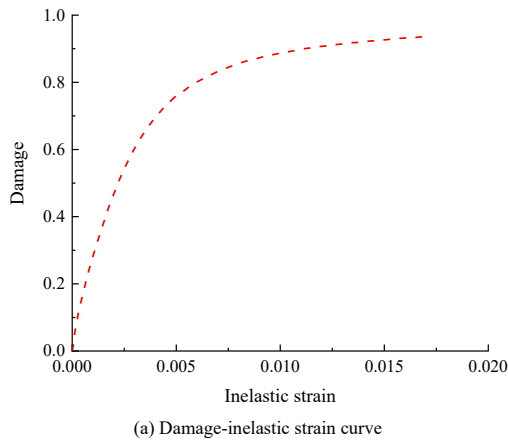
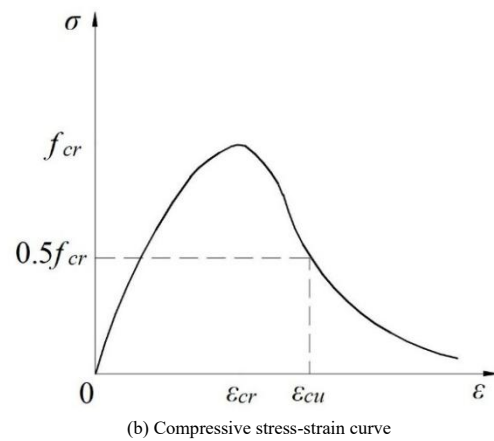


Fig. 11 Stress-strain curve of steel



(a) Damage-inelastic strain curve



(b) Compressive stress-strain curve

Fig. 12 Concrete elastoplastic model

In ABAUQS software, there are three types of concrete material models, and the concrete plastic damage model is selected for analysis. The damage-inelastic strain curve of concrete is shown in Fig. 12(a), in which the concrete adopts a damage factor that reflects the degradation rate of material stiffness. Concrete damage progresses from 0 (no damage) to 1 (total strength failure) in this quantification system. The constitutive relationship of concrete is given according to the Chinese specification [30], and the stress-strain relationship as shown in Fig. 12(b). In the figure, f_{cr} is the representative value of the uniaxial compressive strength, ϵ_{cr} is the peak compressive strain, and ϵ_{cu} is the compressive strain when the stress is equal to $0.5f_{cr}$.

The applied load of the SGWC beam is the lane load, including the concentrated load applied in the mid-span and the uniform load in the full-span

arrangement. Fig. 13 shows a schematic of the loading of the SGWC beam model with a span length l of 60 m.

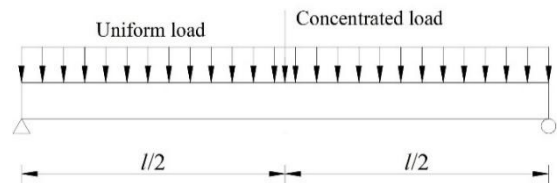


Fig. 13 Schematic diagram of model loading

P_u is defined as the ultimate bearing capacity, and P_u is determined according to the ultimate strength of the SGWC beam. ABAQUS simulated the complete load-to-failure behavior of the SGWC beam. Fig. 14 presents the load-displacement relationship of the SGWC beam under lane loading, derived from mid-span vertical displacement and bearing capacity data.

As can be seen from Fig. 14, the failure process of the SGWC beam consists of three stages: (1) the elastic stage with a load range of $0.0P_u - 0.6P_u$, and the load-displacement curve is basically in a linear rising state. At this stage, the elastic deformation of the SGWC beam occurs under the load, and the structure maintains an ideal elastic state. The steep load-displacement slope demonstrates high structural stiffness. (2) In the elastoplastic stage, the load range is $0.6P_u - 1.0P_u$. In this stage, the concrete produces compressive damage, and the steel beam has plastic strain. The load-displacement curve shows a curve-rising state, and its slope gradually decreases, indicating that the stiffness of the SGWC beam gradually decreases; (3) During plastic deformation, concrete damage accumulates while the steel beam develops significant plastic strain, coinciding with peak load capacity. The load-displacement curve is horizontal, and the bearing capacity remains basically unchanged. The plastic deformation of the structure increases significantly until the SGWC beam fails, and the structure basically loses all its stiffness. On the whole, the failure form of the SGWC beam is flexural failure. The longitudinal stress of the SGWC beam at $1.0 P_u$

(ultimate bearing capacity) is shown in Fig. 15.

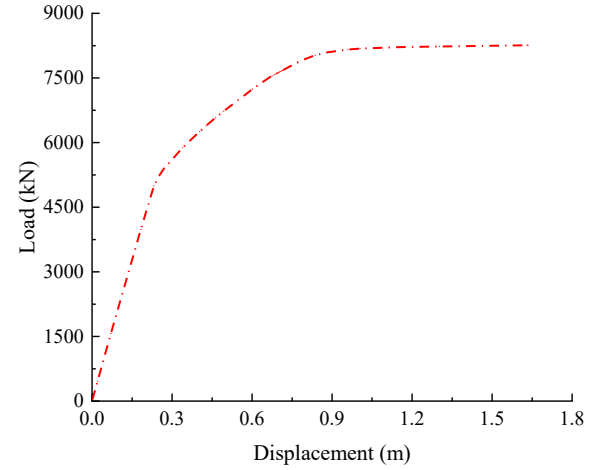


Fig. 14 Load-displacement curves of the SGWC beam

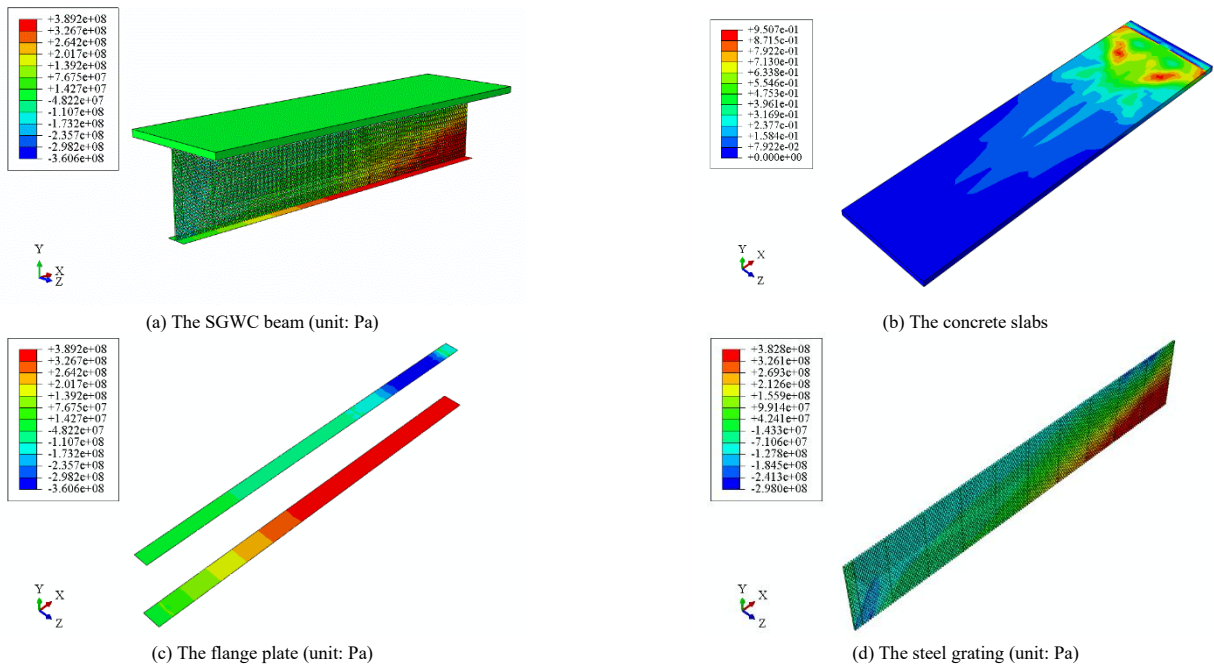


Fig. 15 Stress nephogram of each component of the SGWC beam at $1.0 P_u$

It can be seen from Fig. 15 that when the SGWC beam achieves $1.0 P_u$, the concrete slab is compressed. The concrete damage gradually increased from the bearing to the mid-span, and the damage of the concrete slab near the mid-span was the most serious, with a damage factor of 0.95, but its distribution width was small. The flange plate stress also showed a gradual increase trend from the bearing to the mid-span, and the upper flange plate reached a peak compressive stress of 360.6 MPa. The maximum tensile stress of the lower flange plate is 389.2 MPa, which also occurs in the mid-span section. The stress of the steel

grating is unevenly distributed along the longitudinal and vertical directions, and the maximum stress occurs in the mid-span, but the attenuation rate along the beam end is faster.

To further explore the stress distribution of the steel grating web, the steel grating near bearing, 1/4 span, and mid-span section of the SGWC beam was selected for analysis when the bearing capacity was $1.0 P_u$, and the stress distribution is shown in Fig. 16.

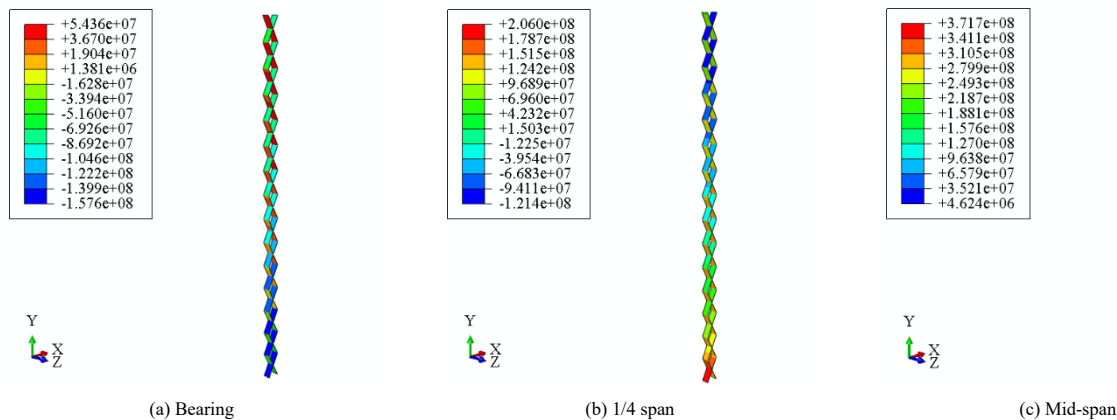


Fig. 16 Stress nephogram of the steel grating with different cross-sections (unit: Pa)

Fig. 16 shows that the upper part of the steel grating near the bearing is in tension and the lower part is compressed. The peak stress of 157.6 MPa develops at the lower flange plate junction, and its upper stress is alternating between tension and compression. The stress distribution of the 1/4 span steel grating is compressed at the upper part and tensioned at the lower part. The maximum stress is tensile stress, which occurs at the junction with the lower flange plate and has a value of 206 MPa. The steel grating near the mid-span is all under tension, and its stress increases rapidly from top to bottom. The tensile stress is larger than that of the bearing and the 1/4 span cross-section, and the maximum value reaches 371.7 MPa.

From the stress distribution and maximum values of the steel grating, the steel grating's tensile stress exhibits gradual growth from the bearing toward mid-span, and the compressive stress near the lower flange plate at the bearing is the largest. The stress of the lower flange plate when the SGWC beam fails is shown in Fig. 17. Except for the contact part between the flange plate and the bearing under the SGWC beam, the rest of the flange plates are all under tension, and the maximum tensile stress in the mid-span is 389.2 MPa. The reason why the SGWC beam could not bear the load further was the destruction of the lower flange plate.

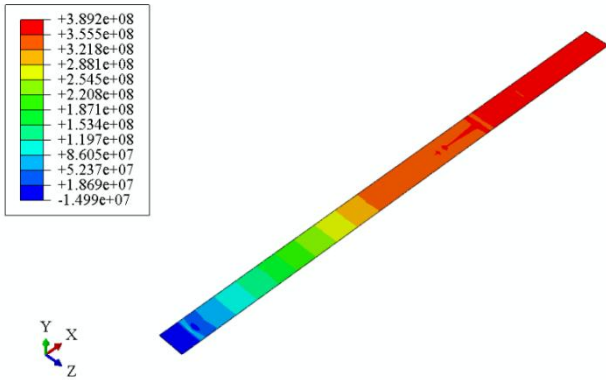


Fig. 17 The lower flange plate under failure of the SGWC beam (unit: Pa)

In summary, the results of elastic-plastic finite element analysis show that the SGWC beam exhibits flexural failure under the lane load. The lower flange plate is the most unfavorable position of the force, and it is the first to reach the ultimate strength and destruction, which leads to the decrease of SGWC beam's bearing capacity. In practical engineering applications, the overall performance of the SGWC beam can be enhanced by using high-strength steel for the lower flange plate, adding stiffeners in the middle span, and incorporating prestressing technology.

5. Parametric analysis

As shear failure must be prevented and the SGWC beam represents a novel structural form, current specifications lack applicable dimensional guidelines. To ensure the bending failure of the SGWC beam, finite element numerical simulation is used in this section to determine the size limit and the influence of size changes of the steel grating under the design size on the failure mode of the SGWC beam. This ensures safe SGWC beam applications and provides design criteria for future engineering practice.

(1) The steel grating spacing k

The steel grating spacing is 150 mm, 200 mm, 250 mm, 300 mm, and 375 mm, respectively, and the remaining parameters of the SGWC beam are kept unchanged. The load-displacement curve under different spacing is drawn as shown in Fig. 18. The figure shows that the bearing capacity of the SGWC beam declines as the steel grating spacing grows. The SGWC beam with a spacing of no more than 300 mm has a relatively obvious plastic deformation stage.

Table 6 presents the SGWC beam's bearing capacity, maximum flange plate and steel grating stresses, and concrete damage states across varying grating spacing. As can be seen from the table, when the steel grating spacing is 375 mm, the maximum stress of the SGWC beam occurs in the steel grating web. Combined with the load-displacement curve of Fig. 18, there is no obvious plastic deformation stage of the load-displacement curve under this spacing, and shear failure occurred in the SGWC beam. With the gradual decrease of the spacing, the maximum normal stress occurs in the flange plate, and the load-displacement curve has an obvious plastic deformation stage. It indicates that the spacing of the steel grating should not be greater than 300 mm.

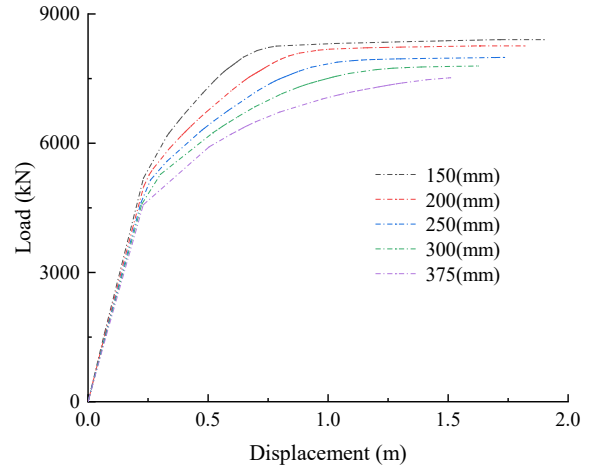


Fig. 18 Variation curve of load-displacement curve with the steel grating spacing

Table 6

Numerical simulation results under different steel grating spacing

k (mm)	P_u (kN)	σ_F (MPa)	σ_W (MPa)	Concrete damage
150	8404.95	397.4	389.9	0.96
200	8258.53	395.8	382.7	0.96
250	7991.34	391.8	387	0.96
300	7792.74	391	385.5	0.96
375	7520.39	377.7	-396.8	0.96

Note: σ_F and σ_W are the normal stress of the flange plate and steel grating, respectively.

(2) The steel grating thickness t

The steel grating thickness starts from 10 mm and increases in 5 mm steps to 30 mm, and the remaining parameters of the composite beam are kept unchanged. The load-displacement curve under different steel grating thicknesses is drawn, as shown in Fig. 19. The figure shows that the bearing capacity of the SGWC beam increases with the rise of the steel grating thickness, and the growth rate of the bearing capacity decreases greatly when the thickness exceeds 20 mm. The SGWC beam with a thickness of not less than 20 mm has a relatively obvious plastic deformation stage.

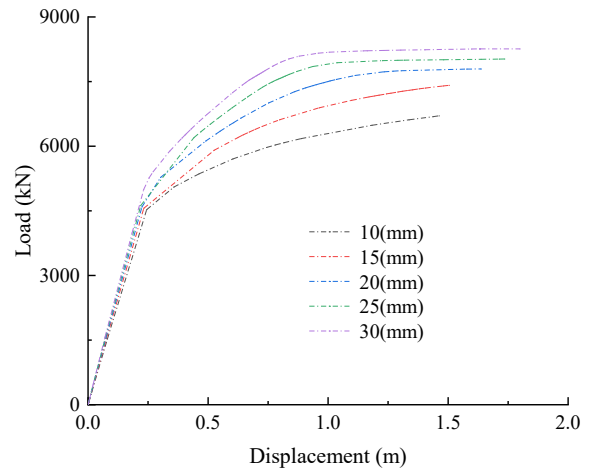


Fig. 19 Variation curve of the load-displacement curve with the steel grating thickness

Table 7 presents the SGWC beam's bearing capacity, maximum flange plate and steel grating stresses, and concrete damage states across varying grating thicknesses. As can be seen from the table, when the thickness is 10 mm, the steel grating web yields first, the flange plate and the concrete still have bearing capacity, and the SGWC beam undergoes shear failure. When the thickness is 15 mm, the flange steel plate experiences peak stress in the composite beam. However, combined with the load-displacement curve of Fig. 19, there is no obvious plastic deformation stage of the SGWC beam at this thickness. As the thickness increases, peak stresses concentrate in the flange steel plate. At this time, the concrete slab loses its bearing capacity, and the load-displacement curve has an obvious plastic deformation stage. It indicates that the thickness of the steel grating should not be less than 20 mm.

Table 7

Numerical simulation results under different steel grating thicknesses

$t(\text{mm})$	$P_u(\text{kN})$	σ_F (MPa)	σ_W (MPa)	Concrete damage
10	6704.56	359	-391.4	0.75
15	7413.98	394.3	384.9	0.88
20	7793.94	391.2	383.7	0.96
25	8024.78	392.2	383.5	0.96
30	8258.53	395.8	382.7	0.96

(3) The steel grating width b

The steel grating width starts from 20 mm and increases in 10 mm steps to 60 mm, and the remaining parameters of the composite beam are kept unchanged. The load-displacement curve under different steel grating widths is drawn, as shown in Fig. 20. The figure shows that the SGWC beam's bearing capacity rises with width, but growth slows significantly beyond 40 mm. The SGWC beam with a width of not less than 40 mm has a relatively obvious plastic deformation stage.

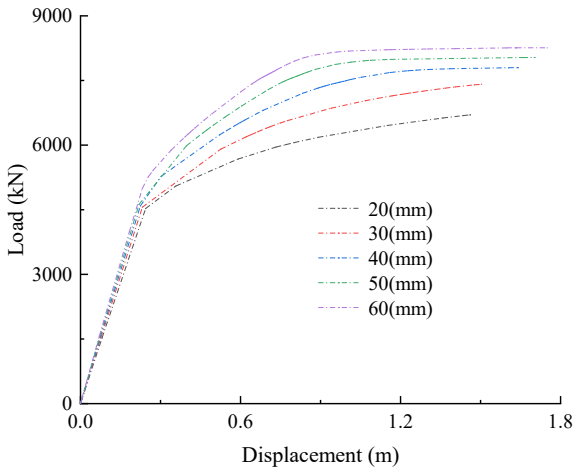
**Fig. 20** Variation curve of the load-displacement curve with the steel grating width

Table 8 presents the SGWC beam's bearing capacity, maximum flange plate and steel grating stresses, and concrete damage states across varying grating widths. As can be seen from the table, when the width is 20 mm, the web of the steel grating is buckled, and the concrete and flange steel plates do not completely lose their bearing capacity, and the SGWC beam structure undergoes shear failure. When the width is 30 mm, the maximum normal stress of the composite beam occurs in the flange plate. The failure modes of the SGWC beam evolve with increasing width as follows: the flange plate yields first, the concrete is crushed, and the load-displacement curve has an obvious plastic deformation stage. It indicates that the width of the steel grating should not be less than 40 mm under the current design dimension.

Table 8

Numerical simulation results under different steel grating widths

$b(\text{mm})$	$P_u(\text{kN})$	σ_F (MPa)	σ_W (MPa)	Concrete damage
20	6703.82	370.5	-396.3	0.75
30	7414.34	390.7	385.9	0.88
40	7797.87	390.6	384.5	0.96
50	8034.03	394.3	383.9	0.96
60	8258.53	395.8	382.7	0.96

6. Discussions

This paper replaces the conventional composite beam web with a steel grating. The linear elastic mechanical properties of the beam are analyzed using the finite element method, followed by elastoplastic analysis to ensure plastic failure criteria are met. The research and development of the SGWC beam aim to address the disadvantages of traditional composite beams, including excessive dead weight and high steel consumption. However, the successful application of the SGWC beam depends on its compatibility with existing bridge construction standards and practices. This is critical for its integration into the current system and widespread use in practical projects.

6.1. Design standard compatibility

The design of the SGWC beam follows the current specifications for steel-concrete composite beams, and the material properties comply with the relevant standards. Linear elastic analysis results show that the SGWC beam complies with Chinese specifications in strength, stiffness, and stability, indicating its suitability for current standards under normal use conditions. Although the current research focuses on the basic stress performance, further analysis of wind-induced vibrations and seismic performance is required per relevant specifications to ensure the safety of the SGWC beam under extreme load conditions.

6.2. Construction compatibility

Existing construction methods, such as prefabricated assembly and cast-in-place, are suitable for the SGWC beams. Given the structural characteristics of the SGWC beam, the prefabricated assembly method offers certain advantages. Commonly used construction equipment in bridge construction is largely applicable. The reduced weight of the SGWC beam allows for optimized equipment selection based on its actual weight, with adjustments mainly needed for parameters like concrete pouring to match the structural characteristics. For specialized construction processes, such as on-site welding of steel grating plates, specialized welding equipment may be required, along with strict control of the welding process to meet quality standards.

6.3. Maintain management compatibility

Existing testing methods, including appearance testing, non-destructive testing, and load testing, are suitable for the SGWC beams, with the addition of key part detection of the steel grating plate. Bridge maintenance and repair can follow existing measures. Conventional measures, such as concrete crack repair and steel structure rust removal and painting, are effective. During maintenance, the special structure of the steel grating plate must be considered, with attention given to hole coating coverage and the inspection of connection parts.

In summary, the SGWC beam is compatible with existing bridge construction standards and practices. However, it requires design, construction, and maintenance optimization to accommodate its unique structural form, ensuring its safe application and long-term performance in engineering projects.

7. Conclusions

This paper proposes a new steel grating web composite beam. Its basic mechanical principle is analyzed, and the feasibility of the structure is checked. The bearing capacity is analyzed by studying its failure mechanism, and the following conclusions are obtained:

(1) The SGWC beam mainly comprises the concrete slab, the steel grating web, and the flange steel plates. The steel grating web is primarily used to bear the shear force, and the concrete slab and flange steel plates are used to bear the bending moment. Based on industry standards and specification requirements, the size range of the SGWC beam is formulated. In addition, in uniaxial behavior, the steel grating exhibits excellent linear elastic characteristics.

(2) The linear elasticity analysis for the 60 m simply-supported SGWC beam is adopted. Its strength, stiffness, and stability meet the specification requirements, and the structure's feasibility is verified. The self-weight can be reduced by 21.5% compared with the traditional steel beam under the current proposed size, and the engineering cost and assembly difficulty are effectively reduced.

(3) The failure mechanism of the SGWC beam was analyzed by elastoplastic finite element, and the failure process consists of three stages, namely the linear elastic stage (0.0 P_u - 0.6 P_u), the elastoplastic stage (0.6 P_u - 1.0 P_u) and the plastic stage. In the whole process of failure, the stress of the lower flange plate first reaches the ultimate strength, resulting in the SGWC beam not being able to continue to bear the load.

(4) The load-bearing capacity of the SGWC beam decreases with the increase of the steel grating spacing and rises with the increase of the thickness and width of the steel grating. To ensure that the SGWC beam does not suffer shear failure under the design size of this paper, the spacing of the steel grating should not be greater than 300 mm, the thickness should not be less than 20 mm, and the width should not be less than 40 mm.

(5) This paper investigates the mechanical performance of the SGWC beam using numerical simulations and analyzes the uniaxial stress test of the proposed steel grating. However, the model test of the composite beam is crucial as it can significantly address the limitations of numerical analysis. Future research will focus on model trials of SGWC beams and their application in continuous beam bridges. Additionally, considering the shear resistance of the web, arranging the

steel grating with variable spacing can significantly optimize the web design. The study of the steel grating web wrapping the concrete to effectively inhibit its buckling and improve the overall bearing capacity will also be carried out.

Declaration of competing interest

The authors declare there are no competing interests.

Acknowledgment

This work was supported by the [Chongqing Joint Training Base for Postgraduate] under Grant [JDLHPYJD2020020].

Data availability statement

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

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UNCERTAINTY QUANTIFICATION OF STEEL TRUSS ASSEMBLY ACCURACY BASED ON ERROR PROPAGATION AND METAMODEL-BASED METHODS

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ABSTRACT

The manufacturing errors of steel truss members exhibit considerable randomness, potentially compromising the on-site assembly accuracy and hindering compliance with the specified assembly requirements. This paper proposes a theoretical analysis approach based on error propagation and quantifies the assembly accuracy with metamodel-based methods to address this issue. Furthermore, the reliability is taken as an indicator to ensure the assembly accuracy due to the stochasticity. First, develop error propagation and accumulation models of statically indeterminate steel trusses through structure deconstruction. Then, determine the uncertainty factor and the assembly accuracy function. Finally, compare the assembly accuracy and efficiency of the Monte Carlo simulation and approximation, simulation, and metamodel-based methods for better subsequent quantification calculation. The assembly accuracy and sensitivity analysis of two different structural steel trusses in practical engineering are conducted. The assembly accuracy results show that the assembly accuracy of the first steel truss is 1.200mm, 1.239mm, 1.053mm, and 0.988mm, and that of the second one is 1.293mm, 1.293mm, 1.088mm, and 1.010mm, all with 100% reliability. The sensitivity analysis results indicate that the angular error of the first steel truss is the most critical variable, and the rod length error of the second one plays a vital role during assembly. In addition, the theoretical research is independent of the materials and structures employed, allowing for its application to more complex structures.

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ARTICLE HISTORY

Received: 25 April 2025
Revised: 4 October 2025
Accepted: 5 October 2025

KEYWORDS

Uncertainty quantification;
Error propagation;
Steel truss;
Assembly accuracy;
Monte Carlo simulation;
Metamodel-based methods;
Error propagation

1. Introduction

Steel truss structures are widely used in civil engineering, such as large-span bridges[1], high-rise buildings[2], stadiums[3], and airports[4]. The manufacturing errors of steel truss members exhibit considerable randomness, potentially compromising the on-site assembly accuracy and hindering compliance with the specified assembly requirements[5].

To ensure successful on-site assembly, physical pre-assembly is extensively adopted to verify the machining accuracy of members in the factory to achieve precision matching. It is used to identify quality problems in connection parts for timely resolution[6]. However, it has several shortcomings, such as high cost and a long construction period[7]. Besides, factors that influence assembly are incompletely considered. For large and spatial steel trusses, only neighboring members, instead of the whole structure, are generally physically pre-assembled[8]. Essential factors affecting assembly quality may, hence, be ignored. As a result, there are many cases in which steel trusses cannot be assembled successfully on-site, even after physical pre-assembly.

Virtual assembly, i.e., trial assembly with physical information in a digital environment, has attained widespread attention in the construction industry and has been increasingly employed to solve the above problems[9]. It has been applied in product design[10, 11], virtual construction simulation[12], assembly process simulation [13], quality checking[14], and virtual assembly systems[15]. Although virtual assembly adopts advanced measurement and digital technologies to reverse model existing members and simulate the assembly process, it still falls short of replacing physical pre-assembly in current engineering practices. The primary reason is the interferences arising from various uncertainties in the three-dimensional laser scanning, inverse modeling, and virtual assembly process, which lead to significant deviations between the virtual and practical assembly outcomes.

In practical engineering, some construction difficulties, such as complex construction conditions, high risk of construction safety, high structural linear requirements, etc., make assembly accuracy challenging to control precisely. Adjustment measures will be adopted in such cases, such as utilizing corner and elevation adjustments, longitudinal deviation adjustments, transverse deviation adjustments, and so on[16]. The main span of the steel truss girder of the Shanghai-Suzhou-Nantong Changjiang River Bridge was accurately closed after longitudinal moving, pulling back from both longitudinal and transverse directions[17]. For rapid and smooth closure of the main bridge steel girder of the Wuhu Changjiang River Rail-cum-Road Bridge with high precision, adjusting the cable forces of some stay cables, pushing and longitudinally moving girder sections on the tops of pylon piers, and adjusting temperature differences are employed[18]; During the erection of a three-main-truss steel

girder of the Shanghai-Suzhou-Nantong Changjiang River Rail-cum-Road Bridge, when the diagonal rod position is inaccurate, adjustment measures such as tensile, roofing, and counter-pressure were adopted to complete the rod buttressing[19]. As proved in previously cited literature, the adjustments can effectively complete assembly for subsequent construction. However, for some geometric defects derived from manufacturing errors, it is hard to precisely decide which parts and what values to adjust because the relationship between the errors and assembly accuracy exhibits ambiguity, resulting in a cumbersome adjustment process. As a result, forced assembly generally is the final option in on-site construction[20, 21]. Thus, internal forces and deformations of statically indeterminate structures will be generated, causing more significant uncertainty and safety risks for the structural performance and the construction process. Therefore, accurately calculating assembly precision based on stochastic manufacturing errors has become an urgent and unresolved challenge during assembly.

This paper proposes a theoretical analysis approach based on error propagation and quantifies the assembly accuracy with metamodel-based methods to address the issue. Furthermore, the reliability is taken as an indicator to ensure the assembly accuracy due to the stochasticity. First, develop error propagation and accumulation models of statically indeterminate steel trusses through structure deconstruction. Then, determine the uncertainty factor and the assembly accuracy function. Finally, compare the assembly accuracy and efficiency of the Monte Carlo simulation and approximation, simulation, and metamodel-based methods for better subsequent quantification calculation. The assembly accuracy and sensitivity analysis of two different structural steel trusses in practical engineering are conducted. Assuming alterations in the shape or geometric parameters of a steel truss structure, the core methodology for analyzing and quantifying approaches used during the calculation process remains the same, which is the paper's fundamental value and key contribution. Therefore, the developed strategy is independent of the materials and structures employed, allowing for its application to more complex structures.

The subsequent sections of the paper are structured as follows: Section 2 provides a detailed description of the error propagation and accumulation modeling process of two statically indeterminate structures. Section 3 outlines the uncertainty quantification, including the uncertainty factor, the assembly accuracy function, and strategies for estimation. Taking two different structural steel trusses in practical engineering as an example, the assembly accuracy and sensitivity analysis are implemented in Section 4. Lastly, in section 5, a comprehensive summary of the key points and contributions of the paper is provided.

2. Error propagation and accumulation model

Steel truss structures, treated as statically indeterminate systems, are often analyzed through finite element simulation in structural engineering applications. However, due to the randomness of errors, repeated evaluations of the simulation process are necessary, rendering it highly time-consuming or even impractical. Error propagation and accumulation models are proposed as an alternative and more affordable method for iteratively analyzing the law of propagation and calculating assembly accuracy, considering the random error distribution. These models are developed by deconstructing statically indeterminate structures into statically determinate ones. The deconstruction assumes that the statically indeterminate structure consists of one start structure, one end structure, and n connecting structures, which can be deconstructed into a static structure by disconnecting $(n-1)$ connecting structures.

Error modeling is divided into three steps. First, develop the coordinate system of the start, connecting, and end structures. Second, identify each member's Denavit-Hartenberg (D-H) parameters according to the coordinate systems, and calculate the coordinate transformation matrix between two neighboring coordinate systems[22]. Finally, calculate the coordinate transformation matrix between the start and the end structure coordinate systems. Accordingly, the coordinate transformation matrix between the $(i-1)$ th and the i -th coordinate systems is shown in Eq. (1). Moreover, assuming that the 0-coordinate system is located at the start structure and the m -coordinate system is located at the end structure, the coordinate transformation matrix between the two structures is calculated by Eq. (2).

$${}^{i-1}T_i = \begin{bmatrix} \cos \theta_i & -\sin \theta_i \cos \alpha_i & \sin \theta_i \sin \alpha_i & a_i \cos \theta_i \\ \sin \theta_i & \cos \theta_i \cos \alpha_i & -\cos \theta_i \sin \alpha_i & a_i \sin \theta_i \\ 0 & \sin \alpha_i & \cos \alpha_i & d_i \\ 0 & 0 & 0 & 1 \end{bmatrix} \quad (1)$$

Where θ_i is the angle from the axis x_{i-1} to x_i counterclockwise rotating in the positive direction of the z_{i-1} -axis; α_i is the angle from the axis z_{i-1} to z_i counterclockwise rotating in the positive direction of the x_i -axis; a_i is the distance from axis z_{i-1} to z_i along the positive direction of the x_i -axis; d_i is the distance from axis x_{i-1} to x_i along the positive direction of the z_{i-1} -axis.

$${}_m^0T = {}_1^0T {}_2^1T \cdots {}_m^{m-1}T \quad (2)$$

Where ${}_1^0T$ is the coordinate transformation matrix between the 0-coordinate and the 1-coordinate systems; ${}_2^1T$ is that between the 1-coordinate and the 2-coordinate systems; ${}_m^{m-1}T$ is that between the $(m-1)$ -coordinate systems and the m -coordinate systems.

2.1. Error propagation modeling of the first statically indeterminate flat truss

Fig. 1 (a) shows the first statically indeterminate flat truss. Assume a bottom chord is the start structure, a top chord is the end structure, and assume a left web member, a diagonal web member, and a right web member are three connecting structures. Suppose the connection between the left web member and the bottom chord and between the right web member and the top chord is disconnected. In that case, the statically indeterminate flat truss is deconstructed into a static determinate one, which is shown as (b) in Fig. 1. Mark the end positions of the members at the disconnection points as A_1 , A_2 , B_1 , and B_2 , respectively. Then, four error propagation models can be developed to investigate error propagation and calculate the coordinates of the points. Since error propagation models contain rod length and angular parameters, l_1 , l_2 , l_3 , l_4 , l_5 , θ_1 , θ_2 , θ_3 , and θ_4 , which are shown as (c) in Fig. 1, the three-dimensional coordinates of points A_1 , A_2 , B_1 , and B_2 , can be accurately calculated through the models, which are under the influence of the rod length and angular errors.

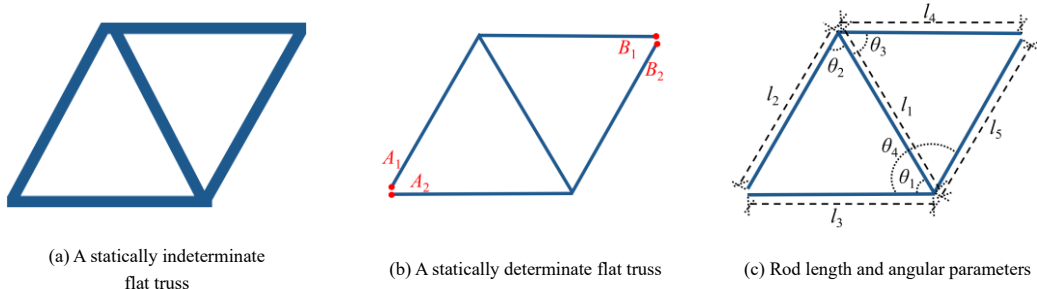


Fig. 1 The first flat truss structure

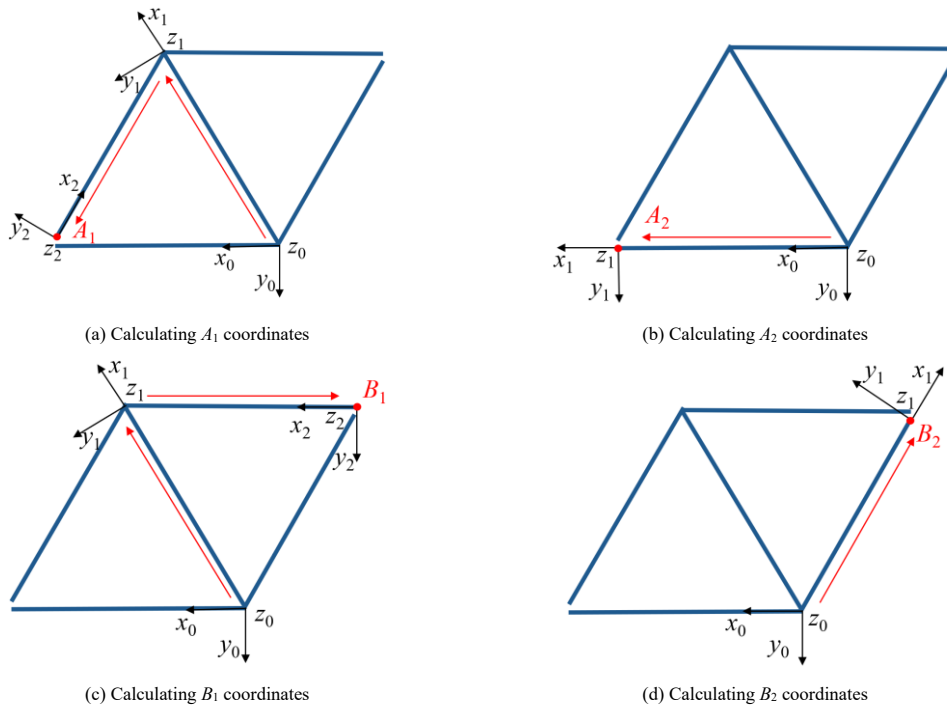


Fig. 2 Error propagation model for calculating the coordinate

As a result, the error propagation models for calculating the coordinates of A_1 , A_2 , B_1 , and B_2 are shown separately in Fig. 2 as (a), (b), (c), and (d).

2.1.1. Error propagation model for calculating the coordinates of A_1

Assume the start and end structures of the error propagation model for calculating the coordinates of A_1 are the 0-coordinate system and the 2-coordinate system, and z_0 , z_1 , and z_2 are oriented perpendicular to the paper plane outward, shown as (a) in Fig. 2. The corresponding D-H parameters, d_i , a_i , α_i , and θ_i , between the 0-coordinate system and 1-coordinate systems are identified as 0, l_1 , 0, and $-\theta_1$, and that between the 1-coordinate system and 2-coordinate system are 0, $-l_2$, 0, and $-\theta_2$. Substitute them in Eq. (1), and the coordinate transformation matrices between the 0-coordinate and 1-coordinate systems and the 1-coordinate and 2-coordinate systems are shown as shown in Eqs. (3) and (4).

$${}^0_1T = \begin{bmatrix} \cos(-\theta_1) & -\sin(-\theta_1) & 0 & l_1 \cos(-\theta_1) \\ \sin(-\theta_1) & \cos(-\theta_1) & 0 & l_1 \sin(-\theta_1) \\ 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix} \quad (3)$$

$${}^1_2T = \begin{bmatrix} \cos(-\theta_2) & -\sin(-\theta_2) & 0 & -l_2 \cos(-\theta_2) \\ \sin(-\theta_2) & \cos(-\theta_2) & 0 & -l_2 \sin(-\theta_2) \\ 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix} \quad (4)$$

Substitute Eqs. (3) and (4) into Eq. (2), the coordinate transformation matrix between the start and end structure coordinate systems of the error propagation model can be calculated, the result of which is shown in Eq. (5).

$${}^0_2T = {}^0_1T {}^1_2T = \begin{bmatrix} \cos(\theta_1+\theta_2) & \sin(\theta_1+\theta_2) & 0 & l_1 \cos \theta_1 - l_2 \cos(\theta_1+\theta_2) \\ -\sin(\theta_1+\theta_2) & \cos(\theta_1+\theta_2) & 0 & -l_1 \sin \theta_1 + l_2 \sin(\theta_1+\theta_2) \\ 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix} \quad (5)$$

From Eq. (5), the coordinates of A_1 are shown in Eq. (6), which represents the effect of the rod length and angular parameters, l_1 , l_2 , θ_1 , and θ_2 , on the coordinates of A_1 during assembly.

$$\begin{cases} x_{A_1} = l_1 \cos \theta_1 - l_2 \cos(\theta_1+\theta_2) \\ y_{A_1} = -l_1 \sin \theta_1 + l_2 \sin(\theta_1+\theta_2) \\ z_{A_1} = 0 \end{cases} \quad (6)$$

2.1.2. Error propagation model for calculating the coordinates of A_2

Assume the start and end structures of the error propagation model for calculating the coordinates of A_2 are the 0-coordinate system and the 1-coordinate system, and z_0 and z_1 are oriented perpendicular to the paper plane outward, shown as (b) in Fig. 2. The corresponding D-H parameters, d_i , a_i , α_i , and θ_i , between the 0-coordinate system and 1-coordinate system are identified as 0, l_3 , 0, and 0. Substitute them in Eq. (1), and the coordinate transformation matrix between the 0-coordinate system and 1-coordinate system is shown in Eq. (7).

$${}^0_1T = \begin{bmatrix} 1 & 0 & 0 & l_3 \\ 0 & 1 & 0 & 0 \\ 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix} \quad (7)$$

From Eq. (7), the coordinates of A_2 are shown in Eq. (8), which represents the effect of the rod length parameter, l_3 , on the coordinates of A_2 during assembly.

$$\begin{cases} x_{A_2} = l_3 \\ y_{A_2} = 0 \\ z_{A_2} = 0 \end{cases} \quad (8)$$

2.1.3. Error propagation model for calculating the coordinates of B_1

Assume the start and end structures of the error propagation model for calculating the coordinates of B_1 is the 0-coordinate system and the 2-coordinate system, and z_0 , z_1 , and z_2 are oriented perpendicular to the paper plane outward, shown as (c) in Fig. 2. The corresponding D-H parameters, d_i , a_i , α_i , and θ_i , between the 0-coordinate system and 1-coordinate system are identified as 0, l_1 , 0, and $-\theta_1$, and that between the 1-coordinate system and 2-coordinate system are 0, $-l_4$, 0, and θ_3 . Substitute them in Eq. (1), and the coordinate transformation matrices between the 0-coordinate system and 1-coordinate system, as well as the 1-coordinate system and 2-coordinate system, are calculated as shown in Eqs. (9) and (10).

$${}^0_1T = \begin{bmatrix} \cos(-\theta_1) & -\sin(-\theta_1) & 0 & l_1 \cos(-\theta_1) \\ \sin(-\theta_1) & \cos(-\theta_1) & 0 & l_1 \sin(-\theta_1) \\ 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix} \quad (9)$$

$${}^1_2T = \begin{bmatrix} \cos \theta_3 & -\sin \theta_3 & 0 & -l_4 \cos \theta_3 \\ \sin \theta_3 & \cos \theta_3 & 0 & -l_4 \sin \theta_3 \\ 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix} \quad (10)$$

Substitute Eqs. (9) and (10) into Eq. (2), the coordinate transformation matrix between the start and end structure coordinate systems of the error propagation model can be calculated, the result of which is shown in Eq. (11).

$${}^0_2T = {}^0_1T {}^1_2T = \begin{bmatrix} \cos(\theta_1-\theta_3) & \sin(\theta_1-\theta_3) & 0 & l_1 \cos \theta_1 - l_4 \cos(\theta_1-\theta_3) \\ -\sin(\theta_1-\theta_3) & \cos(\theta_1-\theta_3) & 0 & -l_1 \sin \theta_1 + l_4 \sin(\theta_1-\theta_3) \\ 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix} \quad (11)$$

From Eq. (11), the coordinates of B_1 are shown in Eq. (12), which represents the effect of the rod length and angular parameters, l_1 , l_4 , θ_1 , and θ_3 , on the coordinates of B_1 during assembly.

$$\begin{cases} x_{B_1} = l_1 \cos \theta_1 - l_4 \cos(\theta_1-\theta_3) \\ y_{B_1} = -l_1 \sin \theta_1 + l_4 \sin(\theta_1-\theta_3) \\ z_{B_1} = 0 \end{cases} \quad (12)$$

2.1.4. Error propagation model for calculating the coordinates of B_2

Assume the start and end structures of the error propagation model for calculating the coordinates of B_2 are the 0-coordinate system and the 1-coordinate system, and z_0 and z_1 are oriented perpendicular to the paper plane outward, shown as (d) in Fig. 2. The corresponding D-H parameters, d_i , a_i , α_i , and θ_i , between the 0-coordinate system and 1-coordinate system are identified as 0, l_5 , 0, and $-\theta_4$. Substitute them in Eq. (1), and the coordinate transformation matrix between the 0-coordinate system and 1-coordinate system is shown in Eq. (13).

$${}^0_1T = \begin{bmatrix} \cos(-\theta_4) & -\sin(-\theta_4) & 0 & l_5 \cos(-\theta_4) \\ \sin(-\theta_4) & \cos(-\theta_4) & 0 & l_5 \sin(-\theta_4) \\ 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix} \quad (13)$$

From Eq. (13), the coordinates of B_2 are shown in Eq. (14), which represents the effect of the rod length and angular parameters, l_5 and θ_4 , on the coordinates of B_2 during assembly.

$$\begin{cases} x_{B_2} = l_5 \cos \theta_4 \\ y_{B_2} = -l_5 \sin \theta_4 \\ z_{B_2} = 0 \end{cases} \quad (14)$$

2.2. Error propagation modeling of the second statically indeterminate flat truss

Fig. 3 (a) shows a different statically indeterminate flat truss structure.

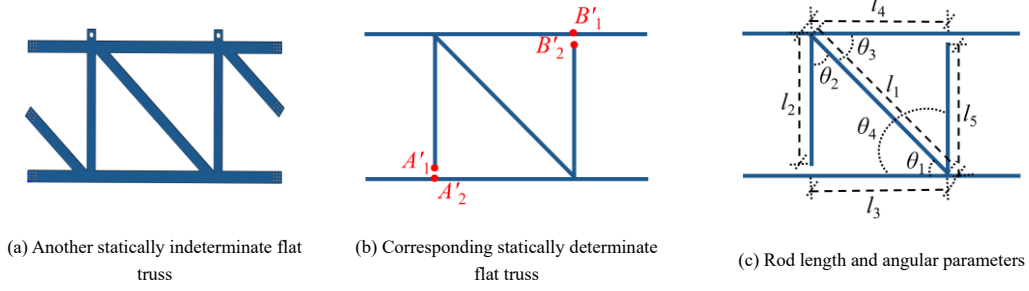


Fig. 3 The second flat truss structure

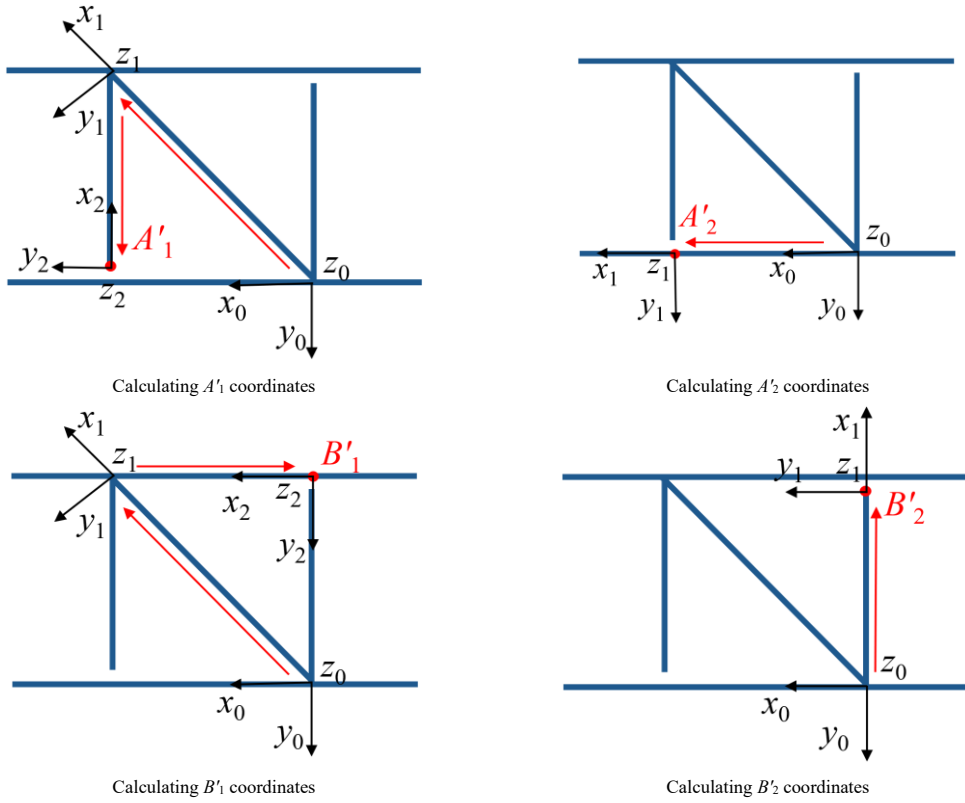


Fig. 4 Error propagation model for calculating coordinates

3. Uncertainty quantification

3.1. Uncertainty factor

All uncertainty factors affecting assembly accuracy during the on-site assembly are represented as rod length and angular errors in the error propagation and accumulation models. Manufacturing error is one dominant factor causing rod length error, and ambient temperature variance is the other. The i -th rod length l_i during assembly is shown in Eq. (15).

$$l_i = (l_{di} + l_{ei}) \times (1 + \alpha \times \Delta T) \quad (15)$$

Where l_{di} is the designed value, mm; l_{ei} is a manufacturing error, mm; α is the linear expansion coefficient, $1/^\circ\text{C}$; ΔT is the ambient temperature variance, $^\circ\text{C}$.

The angular error also resulted from the manual connecting process. The i -th angle θ_i during assembly is shown in Eq. (16). Suppose a rod length is l_m , and the permitted deviation diameter of which is d , its maximum angular error range is $\pm \arctan(d/l_m)$.

$$\theta_i = \theta_{di} + \theta_{ei} \quad (16)$$

Fig. 4 shows the error propagation models for calculating the points A'_1, A'_2, B'_1 , and B'_2 coordinates. The coordinate representation and subsequent calculation are the same as those in Section 2.1.

Where θ_{di} is the designed value, $^\circ$; θ_{ei} is the error value, $^\circ$.

3.2. Assembly accuracy function

Eq. (17) shows the assembly deviation d_A between A_1 and A_2 and d_B between B_1 and B_2 of steel trusses in Sections 2.1 and 2.2. If any d_A or d_B is greater than the designed value of assembly accuracy, the assembly cannot be completed; conversely, it can be completed.

$$d_A = \sqrt{\{[l_1 \cos \theta_1 - l_2 \cos(\theta_1 + \theta_2)] - l_3\}^2 + [-l_1 \sin \theta_1 + l_2 \sin(\theta_1 + \theta_2)]^2} \quad (17)$$

$$d_B = \sqrt{\{[l_1 \cos \theta_1 - l_4 \cos(\theta_1 - \theta_3)] - l_5\}^2 + [-l_1 \sin \theta_1 + l_4 \sin(\theta_1 - \theta_3)]^2 - (l_5 \sin \theta_4)^2}$$

Where l_1, l_2 , and l_5 are the diagonal, left, and right web members of the flat truss; l_4 and l_3 are the top and bottom chords of the flat truss; $\theta_1, \theta_2, \theta_3$, and θ_4 are the angles between the diagonal web member and the bottom chord, between the diagonal web member and the left web member, between the diagonal web member and the top chord, and between the bottom chord and the right web member.

3.3. Strategies for estimation

In the presence of uncertainties regarding the system's physical properties, the structure may operate outside its nominal range. Given all uncertainty factors, the uncertainty analysis aims to quantitatively assess the probability that the structural assembly accuracy on-site satisfies the designed specifications during the assembly process. The Monte Carlo simulation[23] is generally adopted to estimate probability. According to the law of large numbers in statistics, if the samples are large enough, the calculated results of the Monte Carlo simulation will gradually converge to the actual value. Therefore, the paper's sample size is $1e6$ [24, 25]. However, it is often time-consuming because it requires repeated evaluations of the computational model describing the system's behavior.

The approximation, simulation, and metamodel-based adaptive methods are used for calculation and comparison to improve the calculation efficiency. The approximation methods, including First Order Reliability Method (FORM)[26, 27] and Second Order Reliability Method (SORM)[28, 29], are based on approximating the limit-state function locally at a reference point. The simulation methods, such as Importance Sampling (IS)[30, 31] and Subset Simulation (SS)[32], are based on sampling the joint distribution of the state variables and using sample-based estimates of the integral. The metamodel-based adaptive methods, evidenced by Adaptive Kriging Monte Carlo Simulation (AK-MCS)[33, 34] and Bayesian Line Sampling (BLS)[35, 36], are based on iteratively building surrogate models that approximate the limit-state

function in the direct vicinity of the limit-state surface.

4. Case study

4.1. Uncertainty quantification of the first flat truss

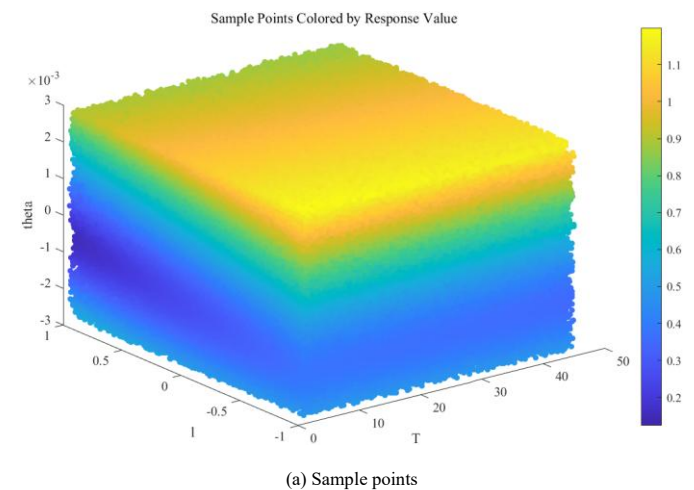
The designed geometric parameters, $l_1, l_2, l_3, l_4, l_5, \theta_1, \theta_2, \theta_3, \theta_4$, are 6042mm, 6042mm, 5000mm, 5000mm, 6042mm, 65.556° , 48.888° , 65.556° , and 114.444° , separately. The manufacturing error of the rod length is ± 1 mm. The permitted deviation diameter of bolts is 0.3mm, which aligns with *Tolerances for fasteners—Bolts, screws, studs, and nuts (GB/T 3103.1-2002)*[37]. Referring to the law of calculating the angular error range in section 3.1, the angular error range of the left, right, and diagonal web members is $\pm 0.0028^\circ$.

After being prefabricated at the factory, steel trusses were transported to the actual construction site. The distance between the two places and the difference in altitude resulted in a significant change in ambient temperature during the manufacturing and assembly. The ambient temperature of the factory is a high temperature of 42°C and a low of -6°C , and that of the construction site is a high temperature of 39°C and a low of -5°C . Therefore, the cooling and warming ranges between the two places are 3 to 47°C and 1 to 45°C . Assuming the rod length and angular error follow a truncated normal distribution[38], different error groups of the flat truss are shown in Table 1. Groups 1 and 2 are set for calculating the assembly accuracy and corresponding reliability between A_1 and A_2 , and 3 and 4 between B_1 and B_2 .

Table 1
Different error groups of the flat truss

Error group	1	2	3	4
Ambient temperature variance range	Warming: 1~45°C	Cooling: 3~47°C	Warming: 1~45°C	Cooling: 3~47°C
Value	± 1 mm		± 1 mm	
Distribution type	Truncated normal distribution		Truncated normal distribution	
Rod length error	Mean value		0	
	Standard deviation		0.333	
	Truncation point		[-1, 1]	
Value	/		$\pm 0.0028^\circ$	
Angular error of the left and right web members	Distribution type		Truncated normal distribution	
	Mean value		0	
	Standard deviation		0.000933	
	Truncation point		[-0.0028, 0.0028]	
Value	$\pm 0.0028^\circ$		$\pm 0.0028^\circ$	
Angular error of the diagonal web member	Distribution type		Truncated normal distribution	
	Mean value		0	
	Standard deviation		0.000933	
	Truncation point		[-0.0028, 0.0028]	

Fig. 5(a) displays the $1e6$ sample points obtained through the Monte Carlo simulation. Fig. 5(b) presents a composite plot illustrating the distribution characteristics of response values, integrating a histogram (grey background), probability density function (PDF, blue curve), and cumulative distribution function (CDF, red curve). Besides, the left Y-axis labelled probability density quantifies the density of the response values within specific intervals, and the right Y-axis labelled cumulative probability indicates the cumulative probability of response values falling below a given threshold, ranging from 0 to 1. The histogram illustrates the frequency distribution of the response values. The PDF provides a smoothed estimate of the probability density across the continuous range of response values, serving as a theoretical fit to the histogram. The CDF shows the cumulative probability of response values up to a specific point, capturing the overall trend of the distribution. The results show that the assembly accuracy corresponding to a cumulative probability density of 100% is 1.2 mm.



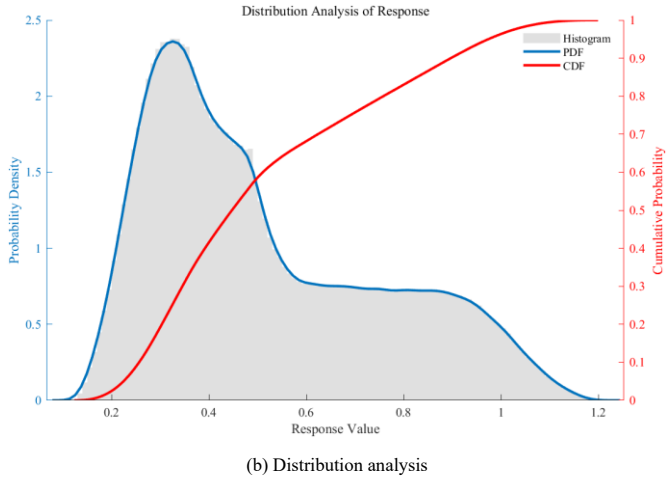


Fig. 5 Sample points and response value using the Monte Carlo simulation

Take the assembly accuracy of MCS as the datum, the FORM, SORM, IS, SS, AK-MCS, and BLS are separately used for better calculation efficiency. Table 2 shows the results of different approaches for Group 1. The $\hat{P}_f$ represents the cumulative probability, and ζ indicates the percentage of relative error of assembly accuracy estimated by the other methods with the datum value, which is shown in Eq. (18). The value's magnitude indicates the technique's estimation accuracy relative to the datum value. Specifically, a smaller value denotes a closer agreement between the estimated result and the datum value, thereby implying higher computational precision. In contrast, a larger value suggests a greater discrepancy between the method's estimate and the datum value, indicating a lower level of accuracy.

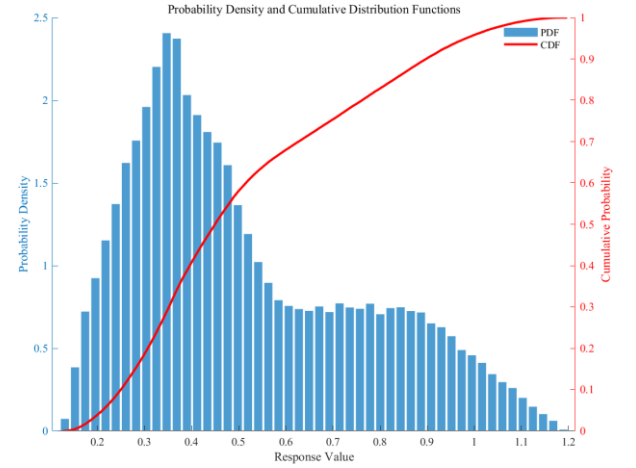
$$\zeta = \left| \frac{P_f - P_{f,MCS}}{P_{f,MCS}} \right| \times 100\% \quad (18)$$

Where P_f is the assembly accuracy of different methods; $P_{f,MCS}$ is the datum assembly accuracy of the MCS.

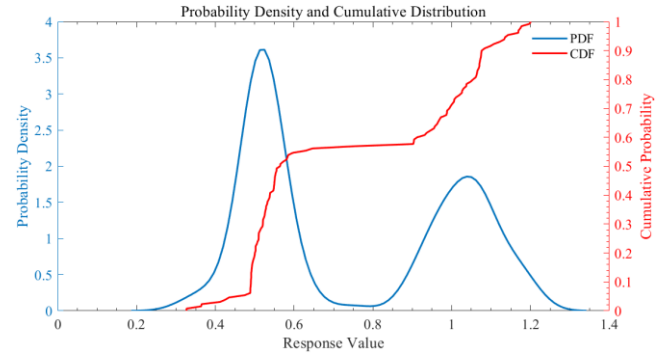
Among all methods, the SS, based on iteratively solving and combining a sequence of conditional reliability analyses utilizing Markov Chain Monte Carlo (MCMC) simulation, has the same assembly accuracy as the MCS with fewer sample points, reduced by a factor of 10. Besides, using a Bayesian active learning framework in combination with line sampling, the BLS has only 0.083% of ζ with sample points of 130, demonstrating exceptionally high computational efficiency and extremely low error. Fig. 6 shows the probability density and cumulative distribution of SS and BLS. It is concluded that SS has the highest accuracy, and BLS maintains exceptionally high accuracy with an extremely low sample size. The BLS perfectly balances sample size and accuracy, ideally suited for high-cost or real-time analysis scenarios. Therefore, it is adopted to calculate the assembly accuracy from Groups 2 to 4, which are 1.239mm, 1.053mm, and 0.988mm, all with 100% reliability.

Table 2
Different approaches for Group 1

Error group	Method	Sample point	Assembly accuracy (mm)	$\hat{P}_f$ (%)	ζ (%)
Datum	MCS	1×10^6	1.200	100	0
Approximation method	FORM	1×10^6	1.198	100	0.167
	SORM	1×10^6	1.204	100	0.333
Simulation method	IS	1×10^3	1.207	100	0.583
	SS	1×10^5	1.200	100	0
Metamodel-based method	AK-MCS	1×10^5	1.195	100	0.417
	BLS	130	1.201	100	0.083



(a) Probability density and cumulative distribution functions of Subset Simulation



(b) Probability density and cumulative distribution of Bayesian Line Sampling

Fig. 6 Probability density and cumulative distribution

Table 3
Different error groups of the spatial truss

Error group	1	2	3	4
Ambient temperature variance range	Warming: 0~46°C	Cooling: 2~48°C	Warming: 0~46°C	Cooling: 2~48°C
Value	$\pm 1.5\text{mm}$		$\pm 1.5\text{mm}$	
Distribution type	Truncated normal distribution		Truncated normal distribution	
Rod length error	Mean value	0		0
	Standard deviation	0.5		0.5
	Truncation point	[-1.5, 1.5]		[-1.5, 1.5]
	Value	/		$\pm 0.001^\circ$
Angular error of the left and right web members	Distribution type	/		Truncated normal distribution
	Mean value	/		0
	Standard deviation	/		0.000333
	Truncation point	/		[-0.001, 0.001]

Angular error of the diagonal web member	Value	$\pm 0.0008^\circ$		$\pm 0.0008^\circ$	
	Distribution type	Truncated normal distribution		Truncated normal distribution	
	Mean value	0		0	
	Standard deviation	0.000267		0.000267	
	Truncation point	[-0.0008, 0.0008]		[-0.0008, 0.0008]	
Assembly accuracy		1.293mm	1.293mm	1.088mm	1.010mm
Reliability		100%		100%	

4.2. Uncertainty quantification of the second flat truss

Similarly, the designed values of rod length and angular parameters, $l_1, l_2, l_3, l_4, l_5, \theta_1, \theta_2, \theta_3, \theta_4$, are 20887mm, 15500mm, 14000mm, 14000mm, 15500mm, $47.911^\circ, 42.089^\circ, 47.911^\circ$, and 90° , separately. The rod length manufacturing error range is ± 1.5 mm. The angular error range of left and right web members is $\pm 0.001^\circ$, and that of the diagonal web member is $\pm 0.0008^\circ$. The cooling and warming ranges are 2 to 48°C and 0 to 46°C .

Table 3 shows different error groups. Groups 1 and 2 are set for calculating the assembly accuracy and corresponding reliability between A'_1 and A'_2 , and Groups 3 and 4 between B'_1 and B'_2 . The results of using BLS to calculate the assembly accuracy are shown in Table 3, and they are all 100% reliable.

4.3. Sensitivity analysis

Let $M(x)$ be an error propagation model, depending on input uncertainty factors, including ambient temperature, length error, and angular error, described by a random vector X . The aim of sensitivity analysis is to investigate how the uncertainty of assembly accuracy is assigned to each input variables X , through $M(x)$. The Sobol' indices[39], predicated on expanding the computational model into summands of increasing dimensionality, serve as reference indices. The Sobol' decomposition is defined in Eq. (19).

$$f(x_1, \dots, x_M) = f_0 + \sum_{i=1}^M f_i(x_i) + \sum_{1 \leq i < j \leq M} f_{ij}(x_i, x_j) + \dots + f_{1,2,\dots,M}(x_1, \dots, x_M) \quad (19)$$

The relative contribution of each group of variables $\{x_{i_1}, \dots, x_{i_s}\}$ to the total variance is represented in Eq. (20). The index concerning one input variable X_i is called the first-order Sobol' index and describes the effect of X_i alone. The multiple-term indices, e.g., S_{ij} , $i \neq j$, are referred to as higher-order Sobol' indices, which are interaction indices that account for the effects of the interactions of X_i and X_j that cannot be decomposed into the contributions of those variables separately[40].

$$S_{i_1, \dots, i_s} = \frac{D_{i_1, \dots, i_s}}{D} \quad (20)$$

Fig. 7 shows the sensitivity analysis results. For the first steel truss in Section 2.1, the First-order Sobol' indices of ambient temperature, length error, and angular error are 0.010755, 0.100913, and 0.893942, as well as the Total-order Sobol' indices are 0.004019, 0.102175, and 0.895793, indicating that the angular error is the most critical variable comparing with ambient temperature and rod length errors. Therefore, focusing on calibrating assembly angles with higher precision can elevate assembly accuracy in such structural practical engineering. Similarly, for the second steel truss in Section 2.2, the First-order Sobol's indices of ambient temperature, length error, and angular error are 0.003955, 0.807207, and 0.177581, and the Total-order Sobol's indices are 0.000007, 0.824145, and 0.192590, representing that the rod length error plays a vital role during assembly.

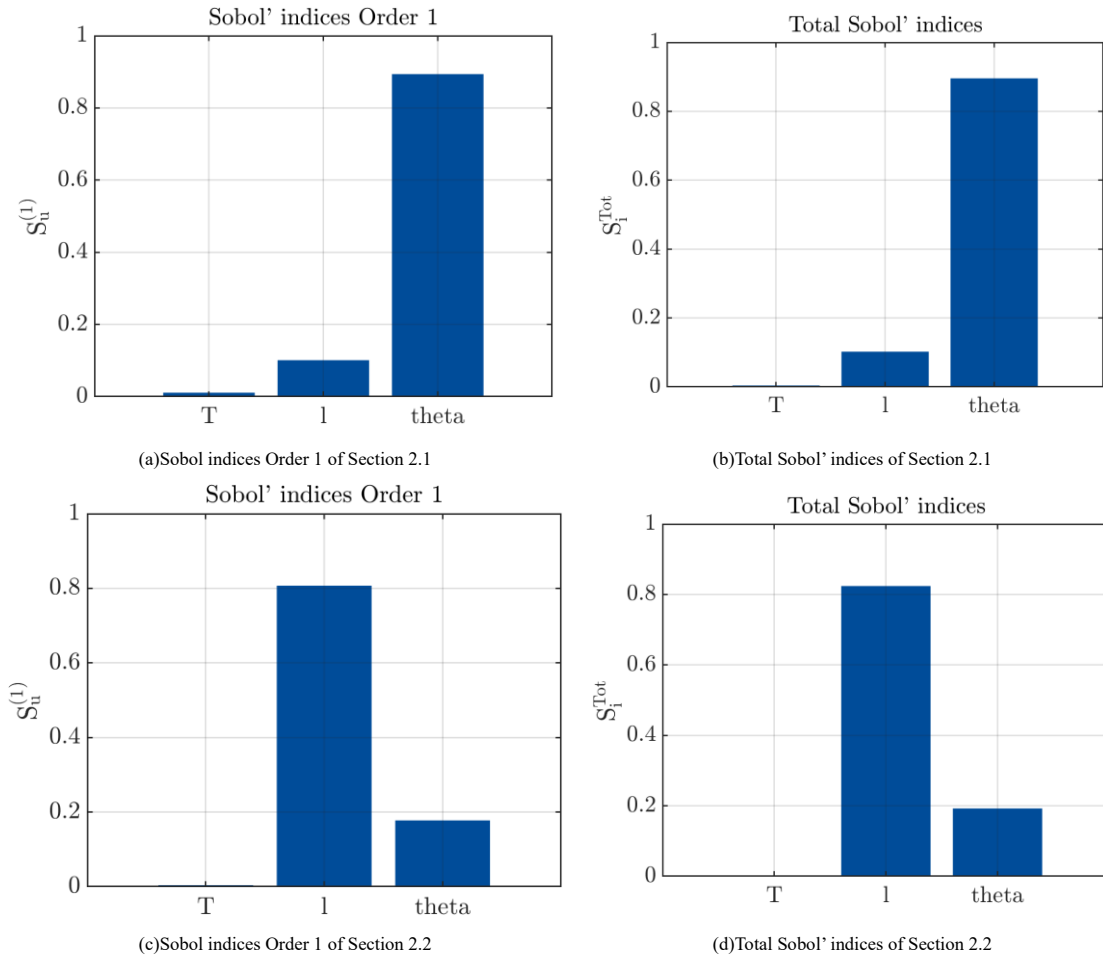


Fig. 7 Sensitivity analysis results

5. Conclusion

This paper aims to quantify the on-site assembly accuracy of members exhibiting random manufacturing errors. The key findings are listed below.

(1) This paper proposes a theoretical analysis approach based on error propagation and quantifies the assembly accuracy with metamodel-based methods. Due to stochasticity, its reliability is used as an indicator to ensure assembly accuracy. The research is independent of the materials and structures employed, allowing for its application to more complex structures.

(2) The error propagation and accumulation models of statically indeterminate structures are developed through structure deconstruction, which has broad applicability in investigating errors with no limitation on various structural figures and geometric dimensions.

(3) Based on the assembly accuracy of MCS, approximation methods including FORM and SORM, simulation methods such as IS and SS, and metamodel-based adaptive methods incorporating AK-MCS and BLS, are used for better calculation efficiency. The results show that SS has the highest accuracy and BLS perfectly balances sample size and accuracy.

(4) The assembly accuracy of the first steel truss is 1.200mm, 1.239mm, 1.053mm, and 0.988mm, and that of the second one is 1.293mm, 1.293mm, 1.088mm, and 1.010mm, all with 100% reliability. The sensitivity analysis results indicate that the angular error of the first steel truss is the most critical variable and the rod length error of the second one plays a vital role during assembly.

Declaration of competing interest

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

Data Availability

Data will be made available on request.

Acknowledgements

The authors acknowledge the financial support from the National Natural Science Foundation of China (No. 52478310).

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STUDY ON SHEAR TEST AND CALCULATION METHOD OF CORROSION COLD-FORMED THIN-WALLED STEEL WITH SELF-DRILLING SCREW-PULL RIVET HYBRID CONNECTION

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ABSTRACT

This paper investigated the effects of corrosion on the shear performance of cold-formed thin-walled steel connections. Two types of connections were studied, specifically the pure self-drilling screw connection and the self-drilling screw-pull rivet hybrid connection. 120 cold-formed thin-walled steel connection specimens accelerated by the outdoor periodic automatic spray salt spray test were subjected to the shear test. The failure mode, shear capacity, and load-displacement curve were obtained. Subsequently, the test results were compared with those of the uncorroded specimens. The results indicated a significant degradation in the mechanical properties of the connection plate due to the corrosion. As the weight loss rate increased, a linear decrease occurred. Under the same weight loss rate, the shear capacity of the pure self-drilling screw connection decreased more than that of the self-drilling screw-pull rivet hybrid connection. This phenomenon became more obvious with the increase of ageing time. When the weight loss rate was 25.1%, the shear capacity of the 0.8 mm pure self-drilling screw connection decreased by 17.89%, while the self-drilling screw-pull rivet connection decreased by only 7.24%. The calculation equation of the shear capacity for pure self-drilling screw connections and self-drilling screw-pull rivet hybrid connections under the condition of corrosion damage were obtained. This calculation provided a reference for determining the shearing capacity of corrosion-damaged connections.

ARTICLE HISTORY

Received: 25 July 2025
Revised: 11 November 2025
Accepted: 12 November 2025

KEYWORDS

Corroded cold-formed thin-walled steel connection;
Shear test;
Shear capacity;
Calculation method

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1. Introduction

Cold-formed thin-walled steel (CFS) structures commonly employ various connection methods, including screws, bolts, rivets, welding, and nails. Among these, self-drilling screws are widely utilized in CFS joint systems and component assembly due to their sufficient stiffness, strength, and deformability. The reliability of connection capacity is crucial for ensuring the overall structural integrity. However, when CFS components undergo significant corrosion under environmental exposure [1-2], degradation in connection strength and deterioration of mechanical properties in steel plates occur [3-7], ultimately compromising structural safety and durability.

Existing research on CFS connections has primarily focused on the shear behavior and capacity of uncorroded specimens [8-10]. Nevertheless, investigations into corroded CFS connections remain limited, despite corrosion being a prevalent environmental factor. The shear performance of such connections is governed by two key aspects: the mechanical properties of the connecting plates and the strength of the fasteners. The formation of irregular corrosion pits during the corrosion process significantly influences the mechanical behavior of steel plates [11-13].

Previous studies on corroded steel and CFS have established fundamental understanding of corrosion mechanisms and mechanical property degradation, yet they differ substantially from the present focus. Si et al. [14-17] demonstrated the dependence of mechanical properties on the complex apparent morphology of corroded steel plates. Zhang et al. [18] investigated fracture behavior in corroded CFS through micromechanical modeling, systematically characterizing the distribution and evolution of fracture parameters. Xu et al. [19] conducted experimental studies on CFS and hot-rolled steel subjected to long-term industrial service, revealing inferior ductility and toughness in corroded CFS compared to hot-rolled steel, with cold-working effects significantly influencing ductility. Qin et al. [20] analyzed the degradation of mechanical properties in corroded steel plates, noting minimal reduction in yield strength, ultimate strength, and fracture strength at mass loss rates below 15%, but significant deterioration beyond this threshold. Roy et al. [21] compared shear performance between corroded and uncorroded self-drilling screw connections, observing substantial strength reduction in corroded specimens. While addressing screw connection corrosion, their work considered neither hybrid connection systems nor comprehensive methods for calculating post-corrosion shear capacity.

The lightweight and flexible nature of CFS building structures, combined with extensive use of self-drilling screws in component-to-joint connections, creates vulnerability to wind-induced vibrations. Such dynamic loading can lead to connection loosening, screw hole enlargement, and eventual fastener pull-out,

resulting in panel detachment from joists. These issues not only reduce load-bearing capacity and affect durability but may also cause severe structural damage upon entering the elastoplastic stage. Furthermore, the thin-walled nature of CFS sections makes them particularly susceptible to bearing capacity deterioration from corrosion, while reduced interlocking action between screw threads and corroded hole walls increases damage susceptibility.

To address these challenges, this study proposes implementing self-drilling screw-blind rivet hybrid connections as an alternative to conventional pure screw connections. Departing from previous research, we designed 30 groups totaling 120 corroded CFS connection specimens considering variables of connection plate thickness, connection type, and aging duration. Through shear tests examining failure modes, shear capacity, and load-displacement relationships, coupled with regression analysis of experimental data, this study establishes calculation methods for shear capacity in both pure self-drilling screw and hybrid connections under corrosion conditions. This approach addresses the limitation of previous single-connection-type studies and provides valuable reference for safety assessment of corroded connections.

2. Experimental investigations

2.1. Test specimen design

Based on the Salt spray test for artificial atmosphere corrosion test (GB/T10125-2012) [22], the specimens were accelerated to a standard outdoor periodic automatic spray salt spray test. The salt fog had a chemical composition of 5% NaCl solution. The specimen was positioned in the testing groove at a 30-degree angle from the vertical direction (Fig. 1). All specimens were connected by connectors and then putted into the corrosion tank. All the steel of the connection was Q235B galvanized sheet. According to the different types of connectors, the specimens were divided into pure self-drilling screw connection (single connection) and self-drilling screw-pull rivet connection (hybrid connection). The self-drilling screw (see Fig. 2a) adopts ST4.2 cross countersunk head type, with a diameter of 4.2 mm. The open flat-headed pull rivet (see Fig. 2b) has a diameter that is approximately 4 mm, which is similar to that of the self-drilling screw. Both the self-drilling screw and the pull rivet are 16 mm in length. Based on the thickness of the connection plate, connection mode, and ageing time, 120 groups corroded CFS connection specimens were designed, each group having at least 4 identical specimens. The dimensions of the specimens are shown in Fig. 3, and the parameters are shown in Table 1.

Table 1
Basic parameters of specimens

Group	Specimen number	Ageing time /day	t/mm	Connection mode
1	2-0.8-d	60	0.8+0.8	single connection
2	4-0.8-d	120	0.8+0.8	single connection
3	6-0.8-d	180	0.8+0.8	single connection
4	2-0.8-h	60	0.8+0.8	hybrid connection
5	4-0.8-h	120	0.8+0.8	hybrid connection
6	6-0.8-h	180	0.8+0.8	hybrid connection
7	2-1.0-d	60	1.0+1.0	single connection
8	4-1.0-d	120	1.0+1.0	single connection
9	6-1.0-d	180	1.0+1.0	single connection
10	2-1.0-h	60	1.0+1.0	hybrid connection
11	4-1.0-h	120	1.0+1.0	hybrid connection
12	6-1.0-h	180	1.0+1.0	hybrid connection
13	2-1.2-d	60	1.2+1.2	single connection
14	4-1.2-d	120	1.2+1.2	single connection
15	6-1.2-d	180	1.2+1.2	single connection
16	2-1.2-h	60	1.2+1.2	hybrid connection
17	4-1.2-h	120	1.2+1.2	hybrid connection
18	6-1.2-h	180	1.2+1.2	hybrid connection
19	2-1.5-d	60	1.5+1.5	single connection
20	4-1.5-d	120	1.5+1.5	single connection
21	6-1.5-d	180	1.5+1.5	single connection
22	2-1.5-h	60	1.5+1.5	hybrid connection
23	4-1.5-h	120	1.5+1.5	hybrid connection
24	6-1.5-h	180	1.5+1.5	hybrid connection
25	2-2.0-d	60	2.0+2.0	single connection
26	4-2.0-d	120	2.0+2.0	single connection
27	6-2.0-d	180	2.0+2.0	single connection
28	2-2.0-h	60	2.0+2.0	hybrid connection
29	4-2.0-h	120	2.0+2.0	hybrid connection
30	6-2.0-h	180	2.0+2.0	hybrid connection



Fig. 1 Specimen corrosion site



Fig. 2 The connector physical diagram: (a) The self-drilling screw; (b) The pull rivet

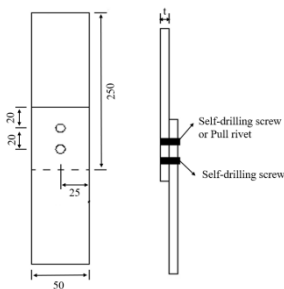


Fig. 3 Connection mode

2.2. Tensile test

Under the erosion of chloride ion (Cl^-), the length of erosion time resulted in different corrosion degrees in the steel connections. The connection plates in the corroded CFS single and hybrid connection specimens with different thicknesses were cleaned using a 12% hydrochloric acid solution, followed by rinsing with water and neutralizing with lime water. Finally, the connection plates were rinsed with water and dried before being cut into 3 standard specimens for tensile test. The standard material specimen size is shown in Fig. 4. The weight loss rate of the specimen could be used as the quantitative index to measure the degree of corrosion. The relationship between the weight loss rate and the ageing time of various steel thicknesses is shown in Fig. 5. It can be seen that under the same ageing time, the weight loss rate increases as the thickness of the connection plate becomes smaller. On the other hand, the thicker the plate, the less prone it is to corrosion. As the ageing time increases, the weight loss rate of connection plates with smaller thicknesses increases significantly. The weight loss rate can be determined using Eq. (1).

$$\eta = \frac{m_0 - m_1}{m_0} \times 100\% \quad (1)$$

where η is the weight loss rate, m_0 is the mass of the specimen before corrosion, and m_1 is the mass of the specimen after corrosion removal and drying.

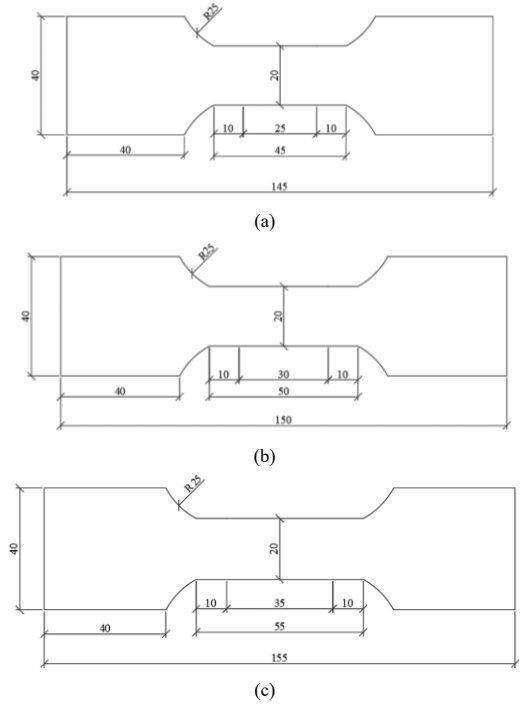


Fig. 4 The standard material specimen size: (a) The thickness of the connecting plate is 0.8 mm and 1.0 mm; (b) The thickness of the connecting plate is 1.2 mm and 1.5 mm; (c) The thickness of the connecting plate is 2.0 mm

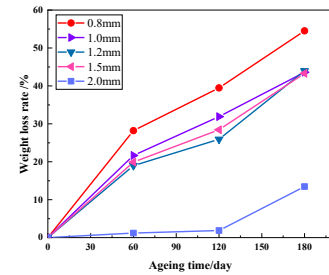


Fig. 5 The relationship between ageing time and weight loss rate of connection plate.

The tensile test was carried out in the universal testing machine. According to Metallic Materials-Tensile testing-Part 1: Method of test at room temperature (GB/T228.1-2010) [23], the loading rate of the elastic stage and the yield stage is 0.75 mm/min. The loading rate of hardening stage is set to 5 mm/min. The mechanical properties of a material can be determined through the tensile test. If there is no clear yield point in the specimen, the stress at 0.2% plastic deformation is considered as the yield strength. Fig. 6 presents the stress-strain curve and Table 2 shows the main performance indicators. The weight loss rate

and main performance indicators are the mean values of the measurement results of the standard specimens of each thickness.

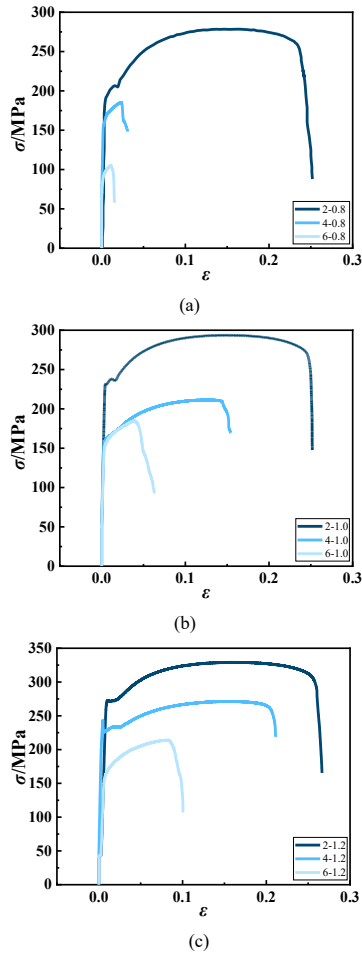


Fig. 6 Stress-strain curves of connection plate specimens in different ageing time: (a) 0.8 mm plate, (b) 1.0 mm plate, (c) 1.2 mm plate, (d) 1.5 mm plate, (e) 2.0 mm plate

As can be seen from Fig. 6, the stress-strain curve of specimens with less weight loss rate still exhibits four typical stages. As the weight loss rate increases, the elastic stage of each specimen becomes shorter and the slope decreases. During the yield stage, the yield platform significantly shortens or disappears due to the stress concentration caused by local etch pits on the surface of the connection plate, and the material enters the plastic stage prematurely. This trend becomes more noticeable as the weight loss rate increases. The peak load during the strengthening stage decreases as the weight loss rate increases. The necking stage also decreases with an increased weight loss rate and disappears once a certain corrosion degree is reached. According to Table 2, the mechanical properties of the connection plates decline as the ageing time increases. Fu et al. [24] studied the mechanical properties of CFS hybrid connection double-limb composite members through experiments. The basic data of uncorroded specimens in this paper refer to this study.

Table 2
Material mechanical test results (Note: 1GPa=1000MPa)

t (mm)	Ageing time (day)	Yield strength (MPa)	Ultimate strength (MPa)	Elastic modulus (GPa)	Weight loss rate (%)	Elongation (%)
0.8	0 [24]	272.5	360.3	200	0	30.7
	60	195.2	277.5	126.31	28.20	25.1
	120	156.3	185.2	104.32	39.47	3.1
	180	81.4	104.7	79.78	54.51	1.5
1.0	0 [24]	259.6	358.2	204	0	32.8
	60	230	293.7	131.4	21.66	25.2
	120	164.2	211.4	99.27	35.76	15.5
	180	152.7	185.1	82.63	43.60	6.3
1.2	0 [24]	288.2	353.4	206	0	29.5
	60	272.5	329.2	142.5	18.98	26.7
	120	225.6	271.3	130.3	25.93	21.1
	180	145.1	214.1	89.9	43.98	10.1
1.5	0 [24]	263.2	355.1	213	0	30.5
	60	235	301.7	140.9	19.98	23.4
	120	202.9	264.6	125.6	28.44	17.9
	180	165.1	212.4	94.9	43.31	16.5
2	0 [24]	262.4	365.8	210	0	31.2
	60	256.2	345.7	199.6	1.20	28.7
	120	244.1	322.8	185.0	1.87	28.0
	180	210.5	280.2	161.4	13.47	19.7

2.3. Degradation rule of mechanical properties

To study the degradation of mechanical properties for connection plates with different ageing times. The weight loss rate was used as a quantitative index for the regression analysis of tensile test value in this paper. The relationship between the yield strength, ultimate strength, elastic modulus, elongation, and the weight loss rate of the steel with different ageing times was obtained (Fig. 7).

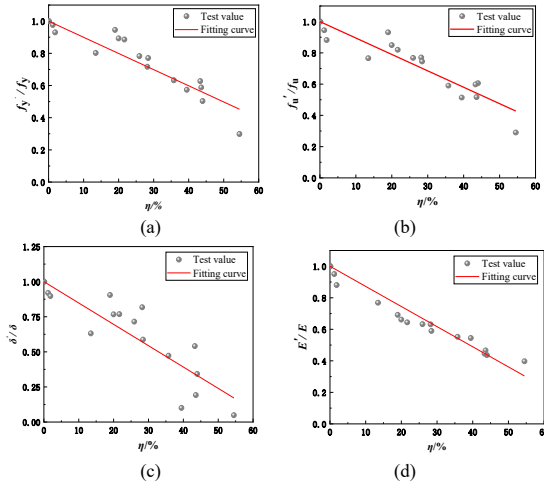


Fig. 7 Relationship between material mechanical and weight loss rate: (a) the yield strength, (2) ultimate strength, (c) elongation, (d) elastic modulus

It can be seen from Fig. 7 that there is an approximately linear relationship between the mechanical properties of the steel and the weight loss rate under different ageing times. The performance indices show different degrees of deterioration as the weight loss rate increases. The mathematical expressions are as follows:

$$f_y'/f_y = 1 - 0.01004\eta \quad (2)$$

$$f_u'/f_u = 1 - 0.0105\eta \quad (3)$$

$$\delta'/\delta = 1 - 0.152\eta \quad (4)$$

$$E'/E = 1 - 0.01277\eta \quad (5)$$

in which, f_y , f_u , δ , E , and f_y' , f_u' , δ' , E' are the yield strength, ultimate strength, elongation and elastic modulus of steel before and after corrosion, respectively.

2.4. Test device and loading system

The loading control and data acquisition were performed by a universal testing machine, and the loading device was shown in Fig. 8. Loading was controlled by the displacement at a rate of 2.5 mm/min. Before loading, the fixtures at both ends were adjusted to ensure that the force of the specimen remained in a straight line.

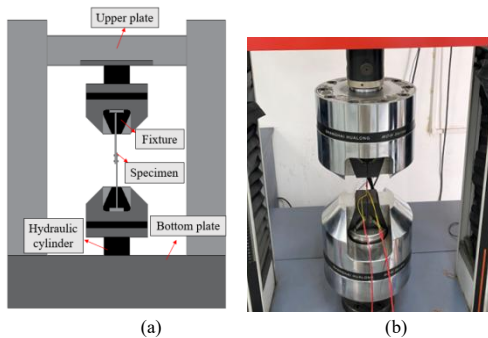


Fig. 8 Schematic diagram of loading device: (a) Loading device schematic, (b) Loading device real shot

3. Test result

3.1. Surface morphology of corroded connection plate

The surface morphology of connection plates before and after corrosion removal in varying ageing times is illustrated in Fig. 9. The corrosion layer on the connection plate, which had been subjected to 60 days of corrosion, was relatively light and thin with an orange hue. Following the removal of the corrosion, the surface of the connection plate exhibited small etch pits. The corrosion layer on the surface of the connection plate thickened and turned orange or reddish brown after 120 days. A slight scrape could remove a fraction of the corroded layer, and a small area of scab was locally distributed on the surface. After removing corrosion, the number of etch pits on the surface of connection plate rose, and certain neighboring etch pits showed a tendency to fuse. After 180 days of corrosion, the corrosion layer on the surface of the connection plate was brown. There was hollowing between the corrosion layer and the base material in certain areas. When touched lightly, the corrosion fell off in large areas. After the removal of the corrosion, most of the nearby etch pits combined to form larger etch pits.

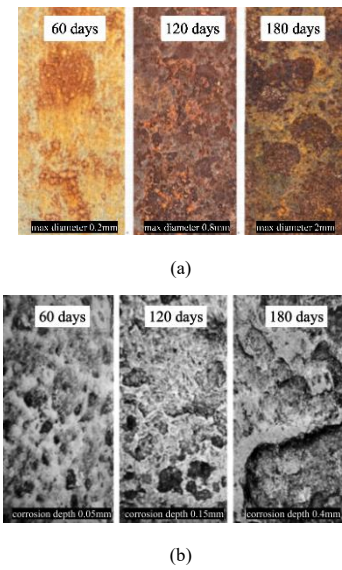


Fig. 9 Surface morphology of connection plate with different ageing times: (a) Before corrosion removal, (b) After corrosion removal

3.2. failure mode

3.2.1. Single connection

The failure modes of the single connection specimens were shown in Fig. 10, respectively. At the initial loading, there was no obvious deformation of the specimen. However, as the load gradually increased, the specimen entered into a plastic stage and the self-drilling screws began to inclined. As the load was increased to the ultimate load, the inclination of the self-drilling screw was exacerbated, and the self-drilling screw of some specimens appeared to be gradually stripped. Concurrently, the specimen emitted a sound of “da da da”. In the final stage of the test, the self-drilling screws were pulled out, the nail holes were enlarged, and the plate connection failed. The aging time was 180 days, and the thickness of the connecting plate was 0.8mm and 1.0mm. When the specimen was destroyed, it was more likely to tear the connecting plate and cause it to be unable to continue to bear the load.

The failure modes of the uncorroded single connection specimen [24] were primarily as follows: (1) When the thickness of the connecting plate was thin, the screw became inclined with the loading and was quickly pulled off due to the enlargement of the pressure hole in the plate. (2) The screw was pulled off during shear process, resulting in a decrease in bearing capacity. However, deformation of the connecting plate was relatively small. (3) The 2.0mm connection specimen experienced very little deformation, and ultimately had its screw cut off.

The change in failure mode of the connecting specimen before and after corrosion could be attributed to several factors including thinning of connecting plate thickness, deterioration of mechanical properties, reduction in diameter of connecting piece, and enlargement of nail hole.

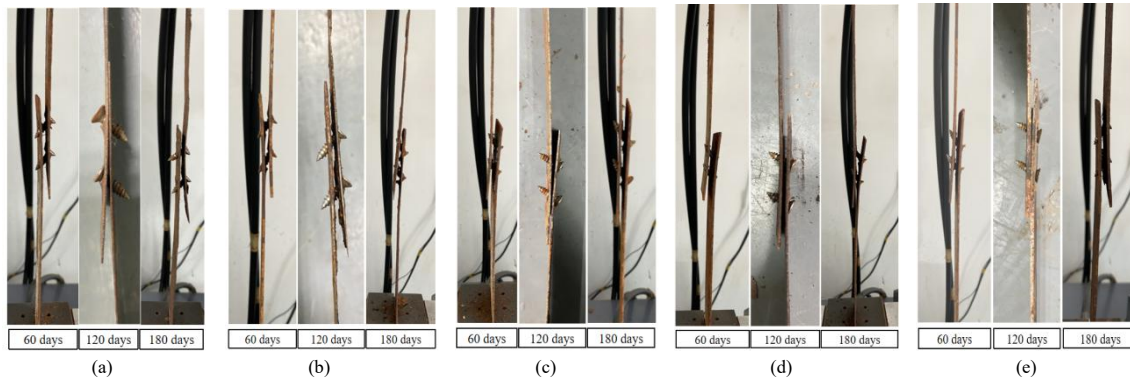


Fig. 10 Single connection failure mode: (a) 0.8mm; (b) 1.0mm; (c) 1.2mm; (d) 1.5mm; (e) 2.0mm

3.2.2. Hybrid connection

The failure modes of the hybrid connection specimens were illustrated in Figure 11, respectively. During the initial loading stage, the hybrid connection specimens exhibited the same mechanical behaviors as the single connection specimens with increasing load. For specimens with connecting plate thicknesses of 0.8 mm, 1.0 mm, and 1.2 mm, the inclination angle of the self-tapping screws increased significantly. Among these, specimens with an aging time of 120 days and 180 days showed a notable increase in the spacing between connecting plates, which was attributed to the enlargement of screw hole diameters caused by corrosion and the mechanical stress acting on the surrounding plate surfaces. It is noteworthy that the inclination angles of specimens with connecting plate thicknesses of 1.5 mm and 2 mm were lower than those of the aforementioned specimens. This is because the larger thickness of the connecting plates, combined with the presence of blind rivets that firmly locked the connecting plates, restricted the inclined deformation of the screws to a certain extent. At the ultimate load, the overall performance of the specimen was characterized by the bearing failure on the surface of the connecting plate surrounding the nail hole and the warping of the plate end. According to the final failure mode diagram, the inclination of self-drilling screws and pull rivets increased significantly in the specimens with an aging time of 120 and 180 days

compared to those with an aging time of 60 days. Additionally, the spacing between connecting plates also increased.

The hybrid connection specimens with connecting plate thicknesses of 1.5 mm and 2 mm showed no obvious deformation at the initial loading stage. As the load increased, the screw gradually inclined and the end of the plate began to warp. Notably, its inclination was lower than that of the hybrid connection specimens with connecting plate thicknesses of 0.8 mm, 1.0 mm, and 1.2 mm. This is due to the connection plate thickness is large, and the presence of pull rivets firmly locked the connection plate, to a certain extent, to limit the screw inclined deformation. At the ultimate load, connecting plate thickness of 1.5mm and 2.0mm specimens produce a "bang" sound, pull rivets were sheared, connecting plate around the nail holes plate surface bearing damage accompanied by the end of the plate warping.

The failure modes of uncorroded hybrid connection specimens [24] can be classified into two primary categories: (1) Hybrid connection specimens with plate thicknesses of 0.8mm, 1.0mm, and 1.2mm exhibited a failure mode in which the connection plate underwent pressure failure, accompanied by screw tilt failure. (2) Specimens with plate thicknesses of 1.5mm and 2mm exhibited a failure mode in which the pull-rivets were cut off, and the deformation of the specimens was small.

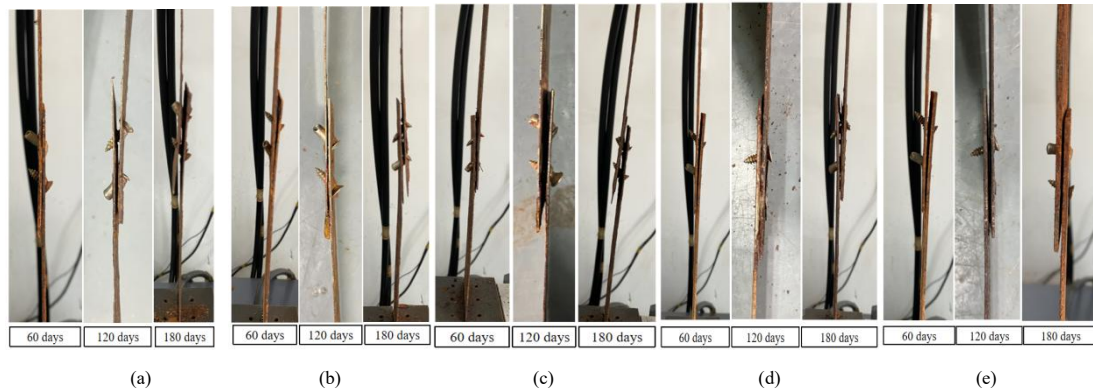


Fig. 11 Hybrid connection failure mode: (a) 0.8mm; (b) 1.0mm; (c) 1.2mm; (d) 1.5mm; (e) 2.0mm

3.3. Load-displacement curve

The load-displacement curve is shown in Fig.12. Overall, the failure process of the connecting specimens had certain common characteristics. During the initial stage of loading, the load-displacement curves of each specimen were linear. The displacement of the connecting specimens with different ageing times did not change significantly with increasing load. However, once the load exceeded a critical limit, a sudden displacement occurred, which became more evident with longer ageing times. This was caused by the brittle bonding effect of corrosion products between the self-drilling screw, the pull rivet, and the connection plate. When the load was below a certain value, the self-drilling screw and the pull rivet bore minimal workload. Nevertheless, when the bonding effect failed, the connector was forced to bear the load.

When the load continues to increase, the curve gradually transits from linear to nonlinear. The slope of the curve exhibited increasing trends followed by decreasing trends. As the weight loss rate increases, the etch pits on the surface of the connection plate become more complex, and stress concentration intensifies. This results in a rapid transition of the steel plate into the plastic stage. Consequently, an observable bending of the load-displacement curve

occurred. When the curve entered the plastic stage, the load-displacement curve tended to be gentle, and the deformation of the specimen connection area intensified.

In the latter phase of loading, The connection plate thickness was 0.8mm, 1.0mm and 1.2mm specimen stiffness decreased due to the self-drilling screws' inclination. The interlocking effect between the self-drilling screw's thread and the connection plate's hole wall enabled the specimens to continue supporting the increased external load. However, when the interlocking effect between the self-drilling screw's thread and the plate's hole wall could not resist the increasing external load, the screw slid off the plate. These caused the load-displacement curve to fluctuate in a wavy manner. The performance of the single connection specimen became more pronounced. However, For the specimens with connection plate thicknesses of 1.5 mm and 2.0 mm, the self-drilling screws of the corroded single connection specimens exhibit pull-off failure when the load reached the shear capacity. This is due to the relatively large thickness of the connection plate and the enlarged holes caused by the corrosion. Among them, the thickness of the connection plate for 4-1.5-d and 6-1.5-d decreased due to corrosion. It could be seen that the load-displacement curve also exhibited a more obvious wave-line trend. The pull rivets of the hybrid connection specimens were cut off after reaching the shear capacity, causing a

sudden drop in load and resulting in brittle fracture.

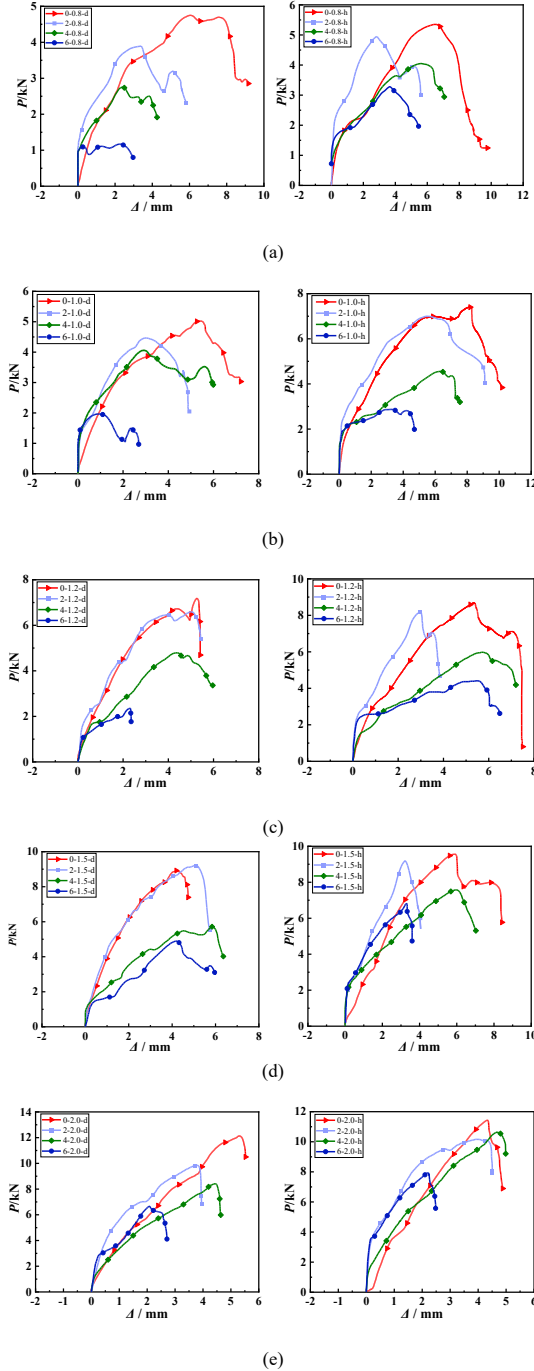


Fig. 12 Load-displacement curve of corroded specimen for different thickness of the connecting plate: (a) 0.8mm. (b) 1.0mm. (c) 1.2mm, (d) 1.5mm, (e) 2.0mm

3.4. Analysis of shear capacity

Table 3 and 4 was the test value of shear capacity with different ageing time. The test results indicate that there was no significant difference between the test values of the uncorroded single connection and the hybrid connection. However, as the ageing time increased, the shear capacity of the single connection specimen with the same thickness decreased considerably when compared to the hybrid connection specimen. This indicated that the self-drilling screws were more prone to corrosion than the pull rivet.

Compared to the uncorroded specimens, the shear capacity for single connection specimens with connection plate thickness of 0.8 mm decreased by 17.89%, 43.15%, and 74.74% respectively, as the ageing time increased. Similarly, the shear capacity of hybrid connection specimens decreased by 7.24%, 24.86%, and 38.78% respectively. For single connection specimens with connection plate thickness of 1.0 mm, the shear capacity decreased by 11.33%, 23.86%, and 60.24% respectively, while that of the hybrid connection specimens decreased by 5.79%, 38.63%, and 60.97% respectively. The shear capacity for single connection specimens with connection plate thickness of 1.2

mm decreased by 8.08%, 33.01%, and 66.57% respectively, and that of hybrid connection specimens decreased by 8.15%, 31.57%, and 51.21% respectively. Moreover, the shear capacity for single connection specimens with connection plate thickness of 1.5 mm decreased by 7.53%, 35.83%, and 45.13% respectively, while that of the hybrid connection specimens decreased by 4.81%, 20.92%, and 37.24% respectively. Finally, the shear capacity for single connection specimens with connection plate thickness of 2.0 mm decreased by 22.98%, 32.95%, and 44.81% respectively, and that of hybrid connection specimens decreased by 10.76%, 17.76%, and 30.88% respectively.

Owing to the galvanization and passivation treatments applied to blind rivets, their corrosion rate in salt spray tests is lower than that of self-tapping screws. In hybrid connections, blind rivets bear part of the shear force, which reduces the stress on self-tapping screws and mitigates the risks of screw hole enlargement and screw pull-out. Additionally, the locking effect of blind rivets on connecting plates inhibits the increase in inter-plate gaps caused by corrosion and alleviates stress concentration. Therefore, compared with single-type connections, hybrid connections are more effective in resisting corrosion-related degradation.

Table 3

Test value of shear bearing capacity of single connection (unit: kN)

Specimen number	0 day [24]	60 days	120 days	180 days
0.8-d	4.75	3.90	2.7	1.20
1.0-d	5.03	4.46	3.83	2.00
1.2-d	7.18	6.60	4.81	2.40
1.5-d	8.93	8.30	5.73	4.90
2.0-d	12.14	9.35	8.44	6.70

Table 4

Test value of shear bearing capacity of hybrid connection (unit: kN)

Specimen number	0 day [24]	60 days	120 days	180 days
0.8-h	5.39	5.00	4.05	3.30
1.0-h	7.43	7.00	4.56	2.90
1.2-h	8.71	8.00	5.96	4.25
1.5-h	9.56	9.10	7.56	6.00
2.0-h	11.43	10.20	9.4	7.90

4. Calculation method of shear capacity

For the connection that uses one self-drilling screw connected, the calculation method outlined in the GB 50018-2002 has been deemed too conservative [25]. As such, the method of calculating shear capacity for hybrid connections utilizes the North American specification [27] to calculate the shear capacity of a single screw connection. At the same time, referring to the calculation method of the shear capacity for the rivet connection in the European standard EN1993-1-3 [28], the rivet part in the hybrid connection is calculated. The sum of the two values is the shear capacity of the hybrid connection (see Eq. 6). The results are shown in Table 5, it can be seen that the calculation results are too conservative, so it is improved.

When analyzing the failure modes of hybrid connections, it is clear that damage to the surface of the connecting plate under pressure leads to deformation of the plate, inclination of the screws, and increased spacing between the plates. This, in turn, leads to an increased risk of pull-off for the screws. However, the rivet can prevent the increase of plate distance, and the rivet has a tendency to cut. It is worth noting that the self-drilling screw offers greater shear strength and can limit the shear of the pull rivet in this context. The shear capacity is influenced by the self-drilling screw and pull rivet connections. When the shear failure of the pull rivet occurs, the pull rivet is cut before the self-drilling screw, and this failure mode mainly occurs in the connection with larger thickness of the composite plate. Based on this, the failure mode is combined with the calculation of shear capacity, and the calculation method of shear capacity under different failure modes after the hybrid connection of rivet and self-drilling screw is modified. When the thickness of the composite plate $t_1 + t_2 < 3\text{mm}$, the shear capacity of the hybrid connection P_h is determined by Eq. (7). When the thickness of the composite plate $t_1 + t_2 \geq 3\text{mm}$, P_h is determined by Eq. (8). The calculation results of shear capacity before and after improvement are shown in Table 5.

Based on the provisions in Section 8 of the North American specification AISI S100-16 [27] for the shear capacity calculation method of a single self-drilling screw connection, and the shear capacity calculation method for riveted

connections specified in the European standard EN 1993-1-3 [28], an improved shear capacity calculation formula for the hybrid joint was proposed:

The shear capacity of the hybrid connection before improvement:

$$P_m = F_b + P_{ns} \quad (6)$$

The shear capacity of the hybrid connection after improvement:

$$P_h = \varphi(F_b + P_{ns}) \quad (7)$$

$$P_h = F_b + \lambda P_{ns} \quad (8)$$

in which, φ and λ are the influence factor coefficient of shear capacity for single screw, taking as $\varphi=1.5$, $\lambda=1.13$ [24], P_{ns} [27], F_b [28].

Table 5
Calculation results of shear capacity for uncorroded hybrid connection

t/mm	P_m	P_h	P_t	P_v/P_m	P_v/P_h
0.8+0.8	3.54	5.28	5.39	1.52	1.02
1.0+1.0	4.92	7.21	7.43	1.51	1.03
1.2+1.2	6.38	9.90	8.71	1.37	0.88
1.5+1.5	8.96	12.58	9.56	1.07	0.76
2.0+2.0	13.21	11.66	11.43	0.87	0.98
Average	-	-	-	1.27	0.93

After comparing the test results and the calculated values in Table 5, it is evident that there is a significant difference between the calculated and test results before the improvement. The average values of the ratio of the calculated value to the test value of the shear capacity of the uncorroded hybrid connection, calculated by the before and after improved method, are 1.27 and 0.93, respectively. The difference between the calculated value and the test value is minimal. It can be concluded that this calculation method is more reasonable than the direct accumulation method.

As the weight loss rate increases, the shear capacity of CFS single connection and hybrid connection gradually decreases. The corrosion of both the plate and the connectors affects the shear capacity, and the degree of corrosion is different. Therefore, the discreteness of the shear capacity value is large. To account for the impact of connector corrosion, we introduce a corrosion factor ε to modify the equation above. The shear bearing capacity of the corroded specimens is P_c . The shear capacity of the connection specimen after thickness reduction is represented by P_t , which is calculated based on the corresponding corrosion rate. The influence factor calculation equation is obtained by fitting the test data. The R^2 of single connection is 0.8899 and 0.9115, respectively. The R^2 of hybrid connection is 0.873 and 0.9789, respectively. The results are shown in Table (6) and Table (7).

$$P_c = \varepsilon P_t \quad (9)$$

Single connection:

$$\varepsilon = \begin{cases} 0.4976e^{0.0394\eta} & \eta \leq 40\% \\ 0.0784e^{0.0784\eta} & \eta > 40\% \end{cases} \quad (10)$$

Hybrid connection:

$$\varepsilon = \begin{cases} 0.6433e^{0.0362\eta} & \eta \leq 40\% \\ 0.0132e^{0.1148\eta} & \eta > 40\% \end{cases} \quad (11)$$

Table 6
Calculation results of shear capacity for CFS single connection

t (mm)	Ageing time (day)	Weight loss rate (%)	P_c (kN)	P_t (kN)	P_c/P_t
0.8	60	28.20	3.52	3.90	0.90
	120	39.47	2.62	2.70	0.97
	180	54.51	1.05	1.20	0.88

1.0	60	21.66	4.17	4.46	0.93
	120	35.76	3.12	3.83	0.81
	180	43.60	2.36	2.00	1.18
1.2	60	18.98	6.39	6.60	0.97
	120	25.93	5.55	4.81	1.15
	180	43.98	3.31	2.40	1.38
1.5	60	19.98	8.72	8.30	1.05
	120	28.44	6.58	5.73	1.15
	180	43.31	4.24	4.90	0.87
2.0	60	1.20	9.16	9.35	0.98
	120	1.87	8.64	8.44	1.02
	180	13.47	6.85	6.70	1.02
Average	-	-	-	-	1.01

Table 7
Calculation results of shear capacity for CFS hybrid connection

t (mm)	Ageing time (day)	Weight loss rate (%)	P_c (kN)	P_t (kN)	P_c/P_t
0.8	60	28.20	4.14	5	0.83
	120	39.47	3.51	4.05	0.87
	180	54.51	2.56	3.3	0.78
1.0	60	21.66	6.16	7	0.88
	120	35.76	5.14	4.56	1.13
	180	43.60	4.5	2.9	1.55
1.2	60	18.98	7.81	8	0.98
	120	25.93	6.88	5.96	1.15
	180	43.98	5.24	4.25	1.23
1.5	60	19.98	9.12	9.1	1.00
	120	28.44	7.32	7.56	0.97
	180	43.31	5.82	6	0.97
2.0	60	1.20	10.00	10.2	0.98
	120	1.87	9.62	9.4	1.02
	180	13.47	7.86	7.9	0.99
Average	-	-	-	-	1.01

From Table 6 and 7, it is evident that, except for individual specimens, the calculation results for the shear capacity of both the single connection and hybrid connection proposed in this paper correspond well with the test results. The ratio of theory to test obtained by the calculation method of shear capacity of single connection and hybrid connection proposed in this paper has an average value of 1.01. It can see that the equations proposed in this paper are consistent and accurate, and the accuracy is high. The proposed calculation method is capable of accurately predicting the shear capacity of both single and hybrid CFS connection specimens, which affected by corrosion.

5. Conclusions

In this paper, the shear test of CFS connection specimens under salt spray corrosion was carried out. The failure mode, shear capacity and load-displacement curve of CFS connection were analyzed. The following conclusions were drawn:

(1) Corrosion causes a clear degradation of the mechanical properties of the connection plate. The yield and tensile strengths decrease linearly with an increase in weight loss rate. Moreover, the elastic stage in the stress-strain curve becomes shorter, the yield point decreases, and the yield platform may even disappear.

(2) The reduction of the shear capacity in pure self-drilling screw connection specimens with various degrees of corrosion is greater compared to that of self-drilling screw-pull rivet hybrid connection specimens. Additionally, the longer the ageing time lasts, the more apparent the phenomenon becomes. For instance, when the weight loss rate is at 25.1%, the reduction rates for the pure self-drilling screw connection with a connection plate thickness of 0.8 mm and the self-drilling screw-pull rivet connection are 17.89% and 7.24%,

respectively.

(3) With the weight loss rate increases, there is no significant difference in the failure mode of the same series of specimens before and after corrosion. The specimens with self-drilling screw-pull rivet hybrid connection still fail due to rivet being cut off and plate bearing failure. The pure self-drilling screw connection specimens fail due to screw pull off and plate bearing failure.

(4) On the basis of the code, considering the corrosion effect of the connector, the corrosion factor is introduced to obtain the calculation equation of the shear capacity of the corroded cold-formed thin-walled steel connection. The ratio of theory to test obtained by the calculation method of shear capacity of single connection and hybrid connection proposed in this paper has an average value of 1.01. It indicates that the proposed method can forecast the shear capacity of pure self-drilling screw connection and self-drilling screw-pull rivet hybrid connection CFS specimens that have corroded more accurately.

6. Application guide for engineering design

Based on the test results and analytical findings, the following recommendations are proposed:

(1) In severe corrosive environments such as coastal areas and high-humidity regions, the self-tapping screw-blind rivet hybrid connection should be prioritized. The thickness of the connecting plate should not be less than 1.5 mm, considering that the bearing capacity of the 1.5 mm-thick plate decreases by 37.24% after 180 days of corrosion, which is significantly lower than the 74.74% reduction of the 0.8 mm-thick plate. (2) In dry and low-corrosion environments, the single self-tapping screw connection can be adopted. (3) Given that a 15% mass loss rate is the mutation point of mechanical properties, inspections should be conducted every 60 days in severe corrosive environments and every 120 days in moderate corrosive environments. When the mass loss rate exceeds 15%, timely replacement or reinforcement measures should be implemented.

Acknowledgment

This study was financially supported by the National Natural Science Foundation of China (Grant No.52268029) and the Basic Research Project of Science and Technology Department of Sichuan Province (Grant no. 2023JDZH0035) and Natural Science Foundation of Sichuan Province (Grant No. 2023NSFSC0381).

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FINITE ELEMENT ANALYSIS ON HYSTERETIC PERFORMANCE OF HOLLOW RIBBED HIGH-STRENGTH CONCRETE FILLED SQUARE STEEL TUBULAR COLUMNS

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ABSTRACT

Hollow ribbed high-strength concrete filled square steel tubular columns offer several benefits, including lightweight, high load-bearing capacity, and ease of installation. Their internal hollow structure also facilitates the arrangement of pipelines in industrial and residential buildings, thereby saving space. This study aims to investigate the hysteretic behavior of these columns under cyclic loads using finite element analysis software, ABAQUS. Initially, the accuracy and applicability of the model established in this study are verified against existing literature. Subsequently, a comprehensive set of finite element models for hollow ribbed high-strength concrete filled square steel tubular columns are developed. These models undergo a full-process analysis, including stress analysis at various critical points. The study examines the influence of parameters such as steel ratio, yield strength of steel tubes, compressive strength of the concrete core, hollow ratio, and axial compression ratio on the hysteretic performance of the columns. Finally, the modified flexural capacity calculation equations are applied to the composite columns in this study and compared with the peak load from the skeleton curve. The findings reveal that the load-displacement curve of these columns can be categorized into three stages: elastic, elastic-plastic, and decline. During the elastic and plastic stages, the steel tubes primarily carry the load, while the internal ribs of the tubes delay member collapse in the decline stage. Based on the parameter analysis, it is recommended that the height of the stiffener should be limited to within 15% of the section width in design. The concrete strength is advised to be maintained below C90. The calculated flexural capacity aligns with the simulated values, with a mean error of 6.7%.

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ARTICLE HISTORY

Received: 1 February 2025
Revised: 11 September 2025
Accepted: 20 September 2025

KEYWORDS

Hollow concrete filled square steel tube;
Stiffening rib;
Flexural bearing capacity;
Hysteresis performance;
Finite element analysis

1. Introduction

The construction industry's ongoing evolution, marked by a surge in large-scale projects like super-tall buildings, long-span structures, and bridges, has highlighted the critical need for research into the performance of structural components under complex loading scenarios, especially seismic forces. Concrete-filled steel tube (CFST) composite structures, representing an innovative approach in structural engineering, have found extensive application in high-rise construction and bridge projects. These structures leverage the strengths of both steel and concrete: the steel tube confers confinement on the core concrete, boosting its load-bearing capabilities, while the concrete inhibits local buckling of the steel tube. This collaborative interaction markedly enhances the structure's overall load-bearing capacity, ductility, and seismic resilience [1-3]. Furthermore, the hollow core of these structures streamlines the installation of pipelines in facilities such as factories and residential buildings, thereby optimizing space utilization. Consequently, this emerging composite structure, grounded in the CFST concept, presents a promising avenue for future structural design and application.

Scholars have conducted extensive research on the fundamental mechanical properties and seismic behavior of hollow CFST columns. Zhang et al[4] conducted numerical analyses using existing experimental data on hollow sandwich CFST columns, concluding that within the parameters of their study, the axial compressive capacity increased with rising concrete compressive strength, thicker inner steel tube walls, and decreasing length-to-diameter ratios and hollow ratios. Yang et al[5] determined the most appropriate concrete constitutive relationship model for hollow CFST short columns under axial compression. Li et al[6-7] developed a theoretical model for the skeleton curve and a calculation method for the displacement ductility coefficient based on quasi-static tests of hollow CFST columns. Ding et al[8] investigated the mechanical properties of circular hollow sandwich CFST superimposed columns under eccentric compression, finding that increasing the yield strength of the steel tube and the nominal steel ratio enhanced the specimen's eccentric compressive bearing capacity. Lu et al[9] researched hollow CFST columns under cyclic loading and established a more precise three-segment recovery force model. Li et al[10] compared the seismic performance of hollow sandwich CFST bridge piers with hollow concrete bridge piers through experimental results and finite element analysis. These studies collectively demonstrate the superior structural benefits and seismic performance of hollow CFST columns. However, they also reveal that when the outer steel tube is thin, it is prone to premature bulging. To mitigate this issue and enhance the steel tube's confinement effect on the concrete, welding stiffening ribs onto the outer steel

tube wall is identified as an effective strategy for improving performance.

Several scholars have investigated the working mechanism of ribbed CFST columns under static and seismic loads. Li et al[11] conducted axial compression tests on 26 thin-walled square CFST short columns to examine their mechanical behavior. The findings indicated that stiffening ribs enhance the bearing capacity of the specimens and slow down the local bulging of the steel tube. An increase in the height of the stiffening ribs correspondingly raised the bearing capacity of the specimens and improved the stability of the steel tube wall. Xing et al[12] performed hysteretic tests on ribbed thin-walled square CFST columns, discovering that the stiffening ribs' constraint on the outer steel tube delayed the local bulging, thereby fully utilizing the material properties of the specimens. Wang et al[13] carried out quasi-static tests on ribbed cold-formed thin-walled square CFST columns, observing a significant improvement in the core concrete's strength due to the constraint of the ribbed cold-formed thin-walled square steel tube. Local buckling of the steel tube occurred post-peak load and was confined to the areas between the longitudinal stiffening ribs and the corners of the steel tube. Through parameter analysis, they developed a hysteretic model for this type of member.

In conclusion, research into the static and hysteretic behavior of both hollow and ribbed CFST columns has advanced significantly. Building on this foundation, the innovative structure of hollow ribbed high-strength concrete filled square steel tubular columns, which merges the benefits of both designs, has increasingly captured the attention of researchers. The hollow configuration not only reduces the structural weight but also enhances the steel tube's confinement effect on the core concrete through the ribbed design. This ribbed structure effectively constrains the transverse deformation of the concrete, slows down concrete cracking and failure, and improves the composite column's performance in terms of bearing capacity, deformation capacity, hysteretic behavior, and seismic resistance [14-15]. However, research on this structural form is still in its nascent stage, lacking comprehensive studies on its working mechanisms and hysteretic behavior. To ensure the seismic safety of buildings utilizing these members, it is imperative to conduct in-depth research on the seismic performance of hollow ribbed high-strength square CFST columns. This research will not only augment existing theories but also provide a crucial foundation for practical engineering design. This paper employs ABAQUS to numerically simulate the behavior of hollow ribbed high-strength concrete filled square steel tubular columns under cyclic loading. It analyzes the complete loading process of typical members and discusses the influence of various parameters, such as steel ratio, steel yield strength, and concrete compressive strength, on the seismic performance of the members.

2. Establishment of finite element models

2.1. Models design

A total of 12 finite element models of members were developed, with their specific parameters detailed in Table 1. A representative member, CFST-1, features a column length of 1200 mm, a square steel tube with a section width of 200 mm, an inner steel tube with an outer diameter of 80 mm, and both the outer and inner steel tubes have a wall thickness of 5 mm. The steel's yield strength is 690 MPa, and the compressive strength of the concrete within the tube column is 80 MPa. Rib plates are welded to the middle position of the outer steel tube's web, measuring 30 mm in height, 10 mm in thickness, and with a yield strength of 690 MPa. The cross-section of this member is depicted in Fig. 1.

Table 1
Model details

Model No	$B \times t_1 \times t_2 \times L/\text{mm}$	F_{y1}/MPa	F_{y2}/MPa	f_{cu}/MPa	h	n
CFST-1	200×5×3×1200	690	690	80	30	0.3
CFST-2	200×5×3×1200	690	690	80	10	0.3
CFST-3	200×5×3×1200	690	690	80	20	0.3
CFST-4	200×5×3×1200	690	690	80	40	0.3
CFST-5	200×5×3×1200	690	690	80	50	0.3
CFST-6	200×5×3×1200	460	460	80	30	0.3
CFST-7	200×5×3×1200	770	770	80	30	0.3
CFST-8	200×5×3×1200	890	890	80	30	0.3
CFST-9	200×5×3×1200	690	690	60	30	0.3
CFST-10	200×5×3×1200	690	690	70	30	0.3
CFST-11	200×5×3×1200	690	690	90	30	0.3
CFST-12	200×5×3×1200	690	690	100	30	0.3

Note: B is the column width, t_1 is the outer steel tube thickness, t_2 is the inner steel tube thickness, L is the column height, f_y is the steel yield strength; f_{cu} is the concrete compressive strength, h is the rib height, n is the axial compression ratio.

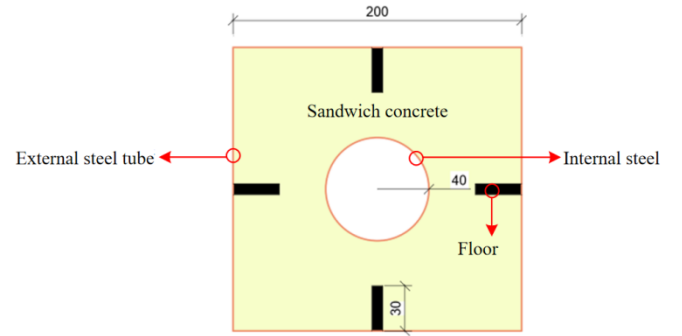


Fig. 1 Cross section design (mm)

2.2. Element types and boundary conditions

Concrete, steel tubes, rib plates, and cover plates were all represented using three-dimensional solid elements with reduced integration and eight nodes (C3D8R). The meshing and boundary conditions are illustrated in Fig. 2. The contact relationships between the components were defined as follows: the cover plate and the inner and outer steel tubes employed “tie” contact, same for the outer steel tube and the rib plates. The cover plate and the concrete utilized a “hard contact”. The normal direction between the steel tube and the sandwich concrete, as well as between the rib plates and the concrete, was set to “hard contact”. The tangential direction employed the Coulomb friction model with a friction coefficient of 0.6 [16].

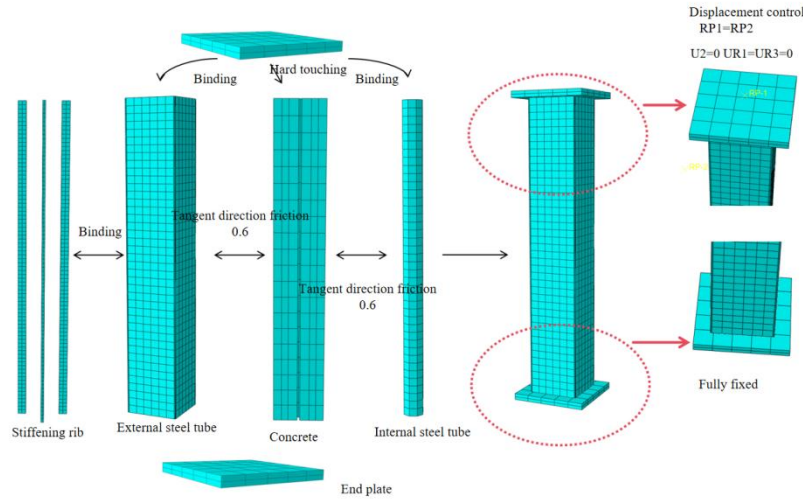


Fig. 2 Boundary conditions

2.3. Constitutive models

In the ABAQUS simulation, the concrete's behavior was modeled using the concrete plasticity damage (CDP) model to capture its constitutive relationship. The stress-strain relationship for the core concrete, which accounts for the confining effect of the steel tube, was based on the model proposed by Liu [17]. The high-strength steel material was represented using a bilinear model [18].

2.4. Model verification

To validate the accuracy of the finite element modeling approach, the method outlined above was applied to model and analyze specimens from literature [19-20]. A comparison of the finite element simulation results with the experimental data from the literature is presented in Fig. 3. The comparison shows that the error in bearing capacity between the finite element and experimental results is within 5%, and the overall curves align closely.

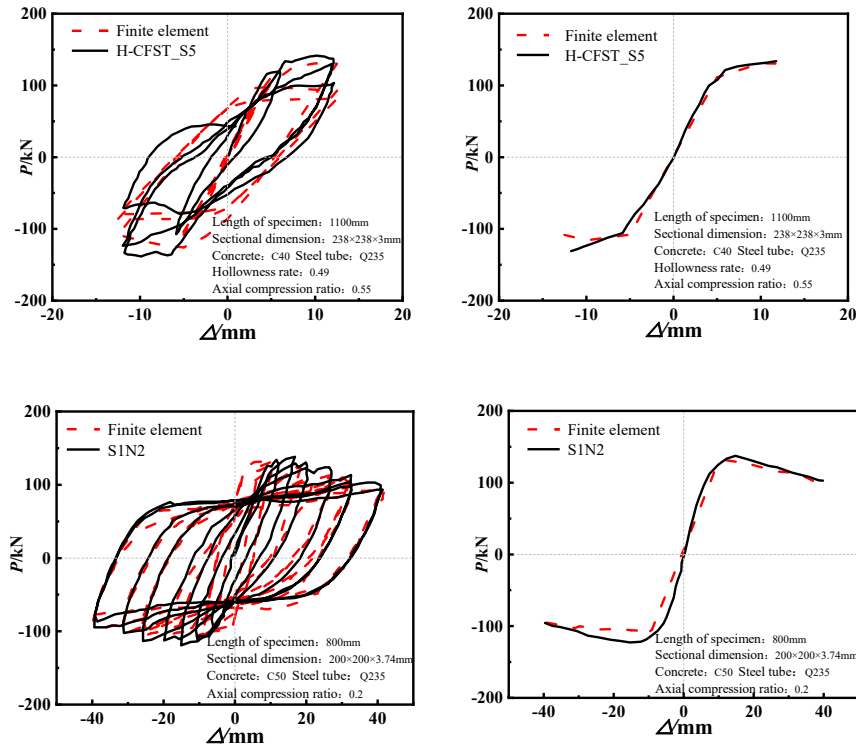


Fig. 3 Model verification

3. Full stress process analysis of typical member

To investigate the seismic performance of hollow ribbed high-strength concrete filled square steel tubular columns, a comprehensive force analysis was carried out on the selected typical member CFST-1.

3.1. Stress process

The load-displacement skeleton curve for the typical member CFST-1 is depicted in Fig. 4. This curve can be categorized into three distinct stages:

Elastic Stage (OA): During the initial loading phase, both the concrete and the steel tubes, both inner and outer, as well as the rib plates, remain in the elastic state. As the horizontal displacement increases, the horizontal load does so linearly, maintaining a constant slope, indicative of a rapid load increase with displacement. The member exhibits a high initial stiffness. In this elastic stage, the stress in the outer steel tube exceeds that in the inner tube, with the stiffening ribs sharing a larger portion of the stress along with the outer tube. At point A, the member's load capacity reaches 87.3% of the peak load, and the outer steel tube in the compression zone is on the verge of yielding.

Elastoplastic Stage (AB): In this stage, the curve continues to ascend. As plastic deformation occurs in the outer steel tube, rib plates, and concrete, the member's stiffness starts to diminish, resulting in a decreasing slope of the curve. The member undergoes overall plastic deformation, transitioning from the elastic to the plastic stage, characterized by a nonlinear relationship between load and displacement. The steel tube's constraint effect on the concrete is significantly enhanced. At point B, the member attains its peak load capacity of 434.2 kN, with the peak displacement being 32.16 mm.

Decline Stage (BC): Beyond the peak point B, the stress in the inner steel tube gradually intensifies and reaches yielding, with the member predominantly in the plastic stage. The load decreases as displacement increases. Following the peak load, the member's load capacity continues to diminish with increasing displacement, accompanied by pronounced bulging of the steel tube. The member is deemed destroyed when the load reaches 85% of the peak load [21].

3.2. Stress analysis

Fig. 5 illustrates the Mises stress for both the inner and outer steel tubes, the stiffening ribs, and the S33 stress of the concrete at the corresponding feature points of the typical member CFST-1.

(1) Feature Point A: At yield point A, the stress in the outer steel tube at the column base approaches the yield stress at approximately 680.1 MPa, while the stress in the inner steel tube remains relatively low at 420.4 MPa. The stiffening

ribs deform in tandem with the outer steel tube, reaching a stress of 684.2 MPa, which underscores their effectiveness. During the initial loading phase, the majority of the concrete is under compression, exhibiting a longitudinal stress of $0.03f_c$ on the tensile side and $1.02f_c$ on the compressive side. This indicates that the steel tube provides confinement to the sandwich concrete.

(2) Feature Point B: At the peak point B, both the outer steel tube and the stiffening ribs have yielded, with the yielded area progressively expanding. However, the stress in the steel tube increases slowly, and the stiffening ribs delay the bulging of the tube. The stress in the inner steel tube also rises, reaching 588.1 MPa at point B. The longitudinal stress on the tensile side of the sandwiched concrete is $0.85f_c$, and on the compressive side, it is $1.55f_c$, suggesting that the confinement effect of the steel tube on the concrete intensifies with increasing load.

(3) Feature Point C: At the ultimate point C, the tensile and compressive zones of the outer steel tube start to merge, causing the deformation to reach its maximum and resulting in the failure of the steel tube. The longitudinal stress on the tensile side of the sandwiched concrete is $0.14f_c$, and on the compressive side, it is $1.85f_c$. Owing to the confinement provided by the inner and outer steel tubes and the rib plates, the concrete demonstrates a higher bearing capacity. The deformation of the rib plates is primarily concentrated at the base of the column, corresponding to the bulging position of the steel tube and the crushing position of the concrete. The inner steel tube yields, reaching 742.7 MPa, and the maximum stress at the base of the column for the outer steel tube and rib plate is 754.8 MPa, achieving the ultimate tensile strength.

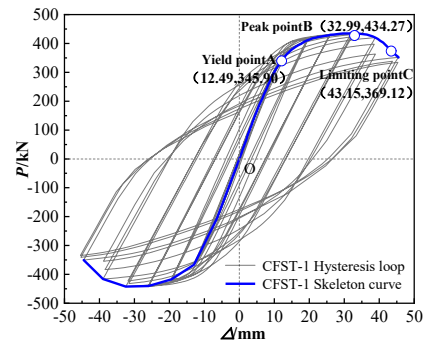


Fig. 4 load-displacement skeleton curve of typical member

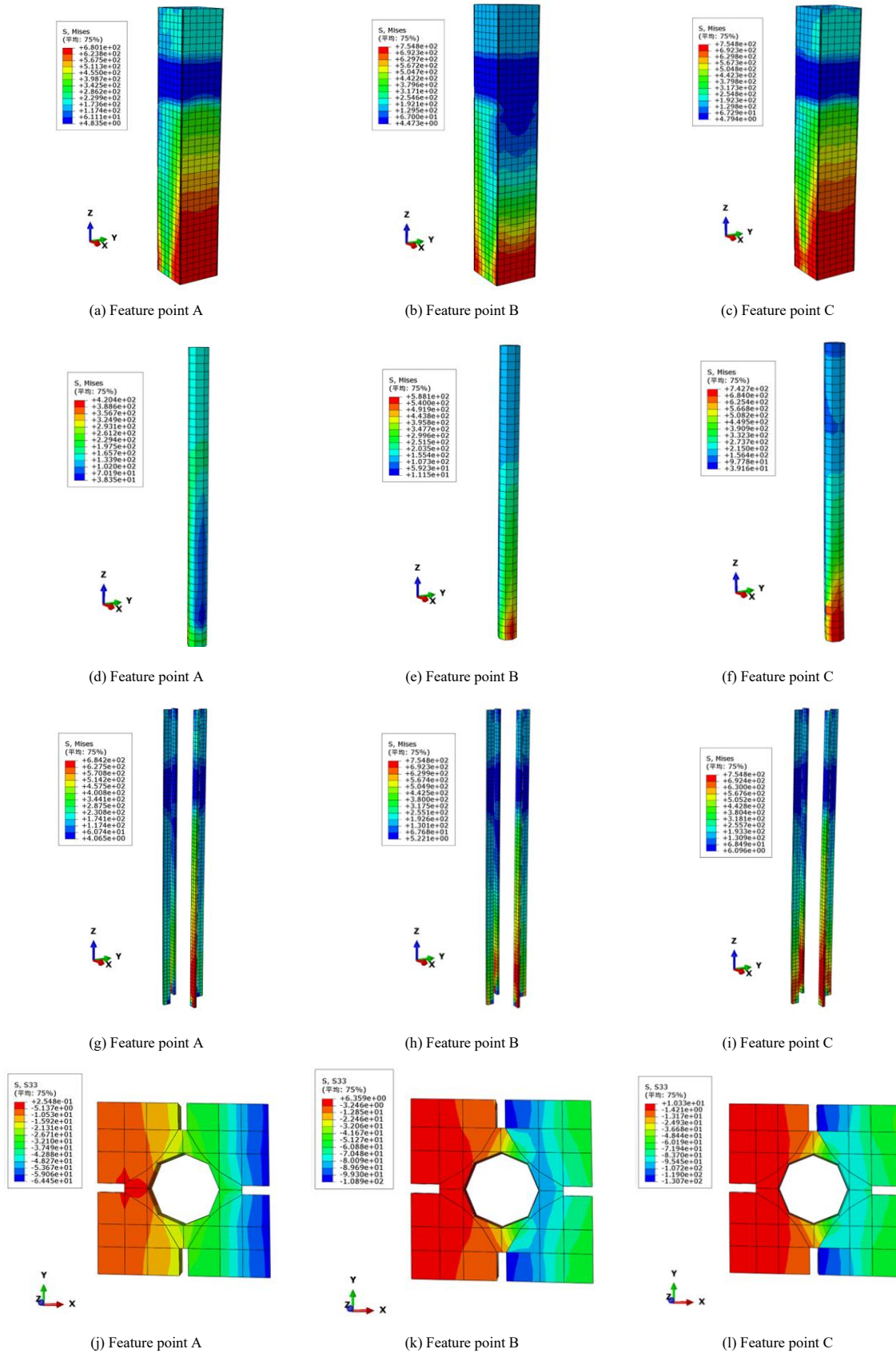


Fig. 5 Stress distribution of components

4. Parameter analysis

The effects of steel ratio (height of stiffening ribs), yield strength of steel tubes, and compressive strength of concrete on the seismic behavior of the members were analyzed.

4.1. Hysteresis curves

Fig. 6 presents the hysteresis curves for each member. As observed in the figure, the hysteresis curves predominantly exhibit a diamond shape, which becomes more filled as the displacement increases. The steel ratio and yield strength of the steel tubes notably influence the shape of the hysteresis curves. The “pinching” phenomenon is generally not pronounced in most of the hysteresis curves, with only a slight occurrence in the hysteresis curve of member CFST-6, where the yield strength of the steel tube is 460 MPa. This suggests that the hollow ribbed high-strength square steel tube concrete columns possess favorable seismic performance. In the initial loading stage, the area

enclosed by the hysteresis loops is small, with no significant residual deformation, and the steel ratio of the member is minimal. As the displacement increases, the area enclosed by the hysteresis loops gradually increases, and the energy-dissipation of the member also increases correspondingly.

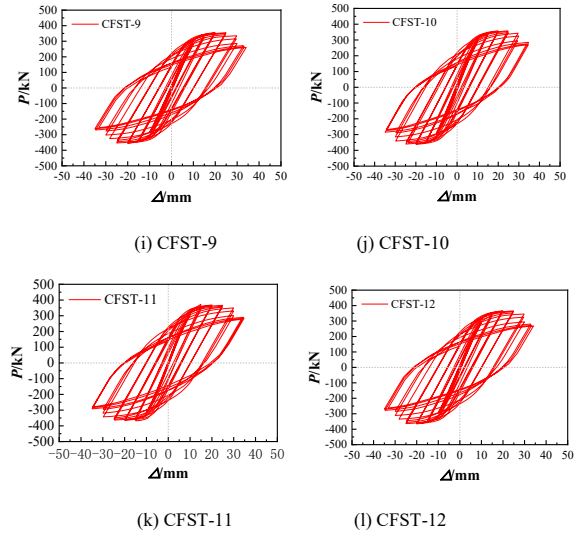
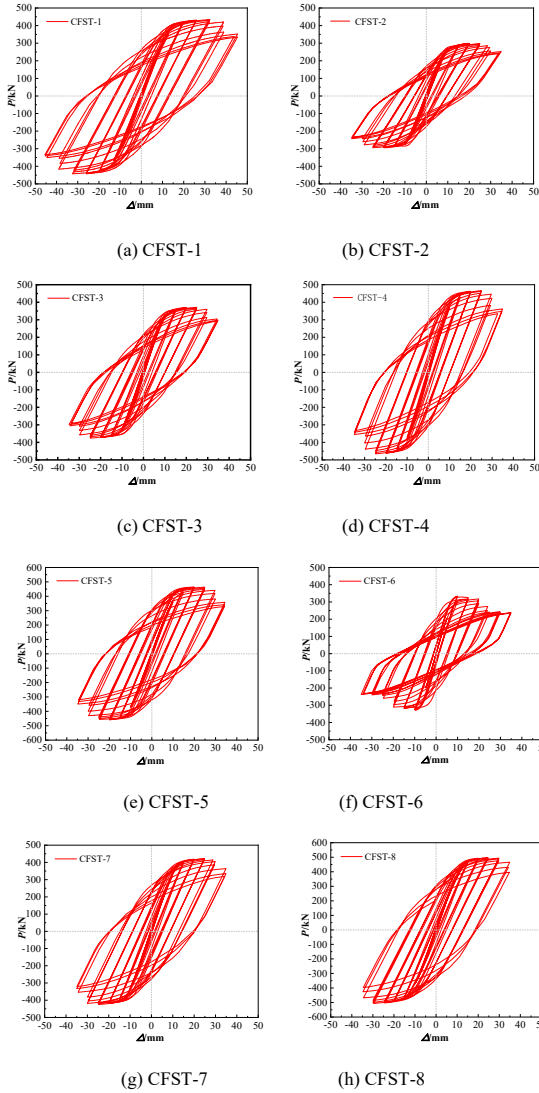


Fig. 6 Hysteresis curves with different parameters

4.2. Skeleton curves

Fig. 7 depicts the influence of parameter variations on the skeleton curves. Fig. 7(a) demonstrates the impact of different steel ratios. It is evident that with an increase in the steel ratio, the initial stiffness of the member progressively enhances, the elastic stage extends, the yield displacement rises, and the peak bearing capacity improves, while the overall shape of the skeleton curve remains largely consistent. For every 1% increase in the steel ratio, that is, for each 10 mm increase in the height of the stiffening ribs from 10 mm to 50 mm, the peak bearing capacity of the member increases by 20.1%, 11.5%, 5.9%, and 3.4%, respectively. This indicates that augmenting the steel ratio can effectively boost the peak bearing capacity of the member, with the most substantial enhancement occurring.

Fig. 7(b) displays the skeleton curves for various yield strengths of the steel tubes. It is apparent that as the yield strength of the steel tubes rises, the elastic stages of the members with different parameters remain largely unchanged, the peak bearing capacity increases, and the descending segment becomes steeper. When the yield strength of the steel tubes is elevated from 460 MPa to 890 MPa, the peak bearing capacity of the members increases by 11.2%, 7.8%, and 5.8%, respectively.

Fig. 7(c) presents the skeleton curves for different compressive strengths of the concrete. It is noticeable that changes in the compressive strength of the concrete have a relatively modest impact on the overall skeleton curve compared to the previous two parameter alterations. There is no substantial difference in the initial stiffness of the members. When the compressive strength of the concrete is raised from 60 MPa to 100 MPa, the peak bearing capacity of the members increases by 3.3%, 1.2%, 2.5%, and 0.6%, respectively.

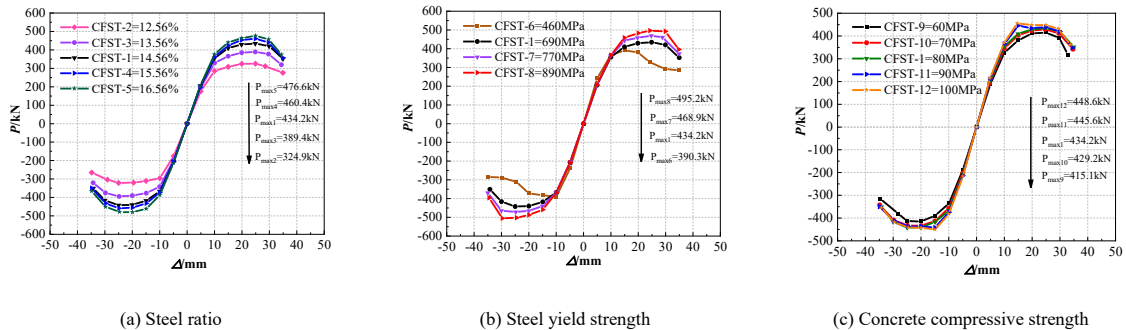


Fig. 7 Skeleton curves with different parameters

4.3. Ductility coefficient

The ductility of the members is determined using the displacement ductility coefficient μ , which is calculated according to Eq. (1) [22].

$$\mu = \frac{|\Delta_u^+| + |\Delta_u^-|}{|\Delta_y^+| + |\Delta_y^-|} \quad (1)$$

In which, Δ_y^+ and Δ_u^+ are positive yield and ultimate displacement, Δ_y^- , Δ_u^- are negative yield and ultimate displacement. The ultimate displacement is the horizontal displacement corresponding to the point where the peak load decreases to 85% of its value; the yield displacement is determined using the equivalent energy method.

Fig. 8 provides insights into how changes in specific parameters affect the skeleton curves of the members. Fig. 8(a) highlights the influence of varying

steel ratios. It is evident that an increase in the steel ratio leads to a gradual enhancement of the member's initial stiffness, an extension of the elastic stage, an increase in yield displacement, and an improvement in peak bearing capacity, while the fundamental shape of the skeleton curve remains largely unchanged. For each 1% increase in the steel ratio, corresponding to a 10 mm increase in the height of the stiffening ribs from 10 mm to 50 mm, the peak bearing capacity of the member increases by 20.1%, 11.5%, 5.9%, and 3.4%, respectively. This indicates that increasing the steel ratio can significantly enhance the peak bearing capacity of the member, with the most substantial improvement observed. Fig. 8(b) illustrates the skeleton curves for different yield strengths of the steel tubes. It can be observed that as the yield strength of the steel tubes increases, the elastic stages of the members with different parameters remain

largely consistent, while the peak bearing capacity improves and the descending segment becomes steeper. When the yield strength of the steel tubes is increased from 460 MPa to 890 MPa, the peak bearing capacity of the members increases by 11.2%, 7.8%, and 5.8%, respectively. Fig. 8(c) shows the skeleton curves for various compressive strengths of the concrete. It is evident that changes in the compressive strength of the concrete have a relatively smaller impact on the overall skeleton curve compared to the previous two parameter changes. There is no significant difference in the initial stiffness of the members. When the compressive strength of the concrete is increased from 60 MPa to 100 MPa, the peak bearing capacity of the members increases by 3.3%, 1.2%, 2.5%, and 0.6%, respectively.

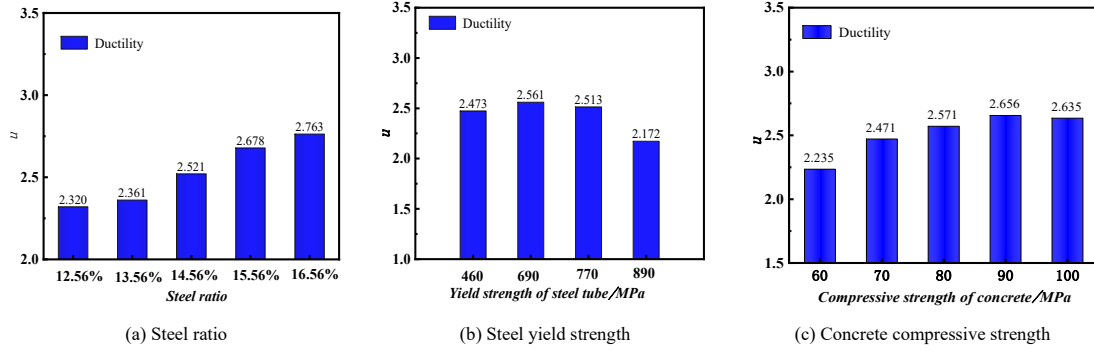


Fig. 8 Ductility coefficient with different parameters

4.4. Stiffness degradation

The stiffness degradation of the members is represented by the secant stiffness K_i , which is calculated according to Eq. (2) [23-24].

$$K_i = \frac{|+P_i| + |-P_i|}{|+\Delta_i| + |-\Delta_i|} \quad (2)$$

In which, P_i is the peak load of the first cycle at the i -th control displacement; Δ_i is the displacement value corresponding to the peak load.

Fig. 9 depicts the influence of parameter changes on the stiffness degradation of the members. In Fig. 9(a), the stiffness degradation curves are presented for different steel ratios. It is evident that a lower steel ratio, which corresponds to a smaller height of the stiffening ribs, results in a relatively faster stiffness degradation rate. As the members approach destruction, the stiffness

degradation rates for members with rib heights of 10, 20, 30, 40, and 50 mm are found to be 80.2%, 79.1%, 77.3%, 75.4%, and 76.3%, respectively. Notably, the member with a steel ratio of 12.56% and a rib height of 10 mm exhibits the most significant stiffness degradation. This observation suggests that increasing the steel ratio can effectively enhance the initial stiffness of the member. Fig. 9(b) illustrates the impact of varying yield strengths of the steel tubes on the stiffness degradation curves of the members. It is observed that the yield strength of the steel tubes has a relatively minor effect on the initial stiffness. The stiffness degradation rates for the members are 84.2%, 77.3%, 75.4%, and 75.6%, respectively, indicating that the stiffness degradation is most pronounced when the yield strength is 460 MPa. Fig. 9(c) displays the effect of concrete compressive strength on the stiffness degradation of the members. It is apparent that as the concrete compressive strength increases, the initial stiffness of the members is greater. However, the overall impact of concrete compressive strength on stiffness degradation is not significant.

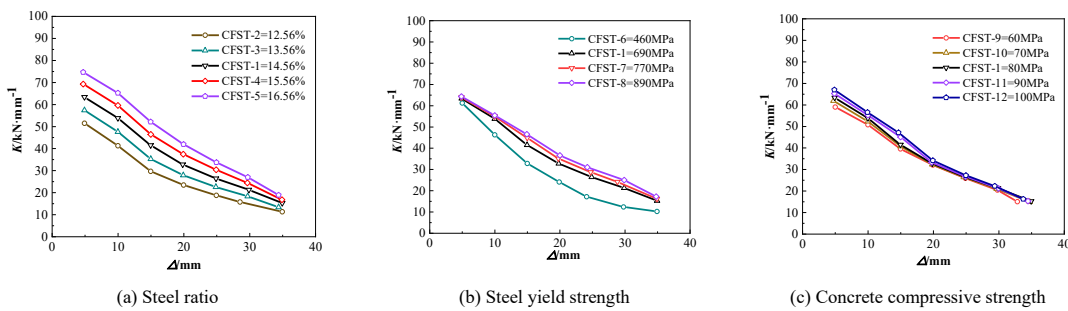


Fig. 9 Stiffness degradation with different parameters

4.5. Energy-dissipation capacity

The energy-dissipation capacity of the members is calculated as the area enclosed by the hysteresis loop in one cycle of the hysteresis curve, and is represented by the energy-dissipation coefficient E , which is calculated according to Eq. (3) [25-26].

$$E = \frac{S_{ABCD}}{S_{\Delta OAE} + S_{\Delta OCF}} \quad (3)$$

In which, S_{ABCD} is the area enclosed by the hysteresis curve; $S_{\Delta OAE}$ and $S_{\Delta OCF}$ are as shown in Fig. 10.

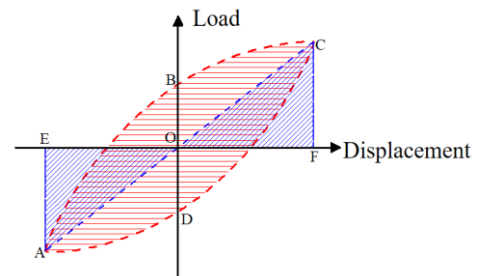


Fig.10 Energy-dissipation calculation

Fig. 11 highlights the influence of parameter changes on the energy-dissipation capacity of the members. In Fig. 11(a), the effect of different steel ratios on the energy-dissipation capacity is illustrated. It is evident that a higher steel ratio correlates with a stronger energy-dissipation capacity for the members. As the steel ratio increases from 12.56% to 16.56%, the cumulative energy-dissipation of the members ultimately rises by 33.1%. Notably, the member with a rib height of 50 mm exhibits the strongest energy-dissipation capacity. Fig. 11(b) presents the energy-dissipation curves for different steel tube yield strengths. The member with a yield strength of 460 MPa demonstrates

a superior initial energy-dissipation capacity, which tends to weaken gradually during the loading process. The general trend observed is that the higher the yield strength, the greater the energy-dissipation capacity of the member. Fig. 11(c) shows the influence of concrete compressive strength on the energy-dissipation capacity of the members. As depicted in the figure, variations in concrete compressive strength have a minimal effect on the energy-dissipation capacity of the members, with the energy-dissipation curves being largely overlapping.

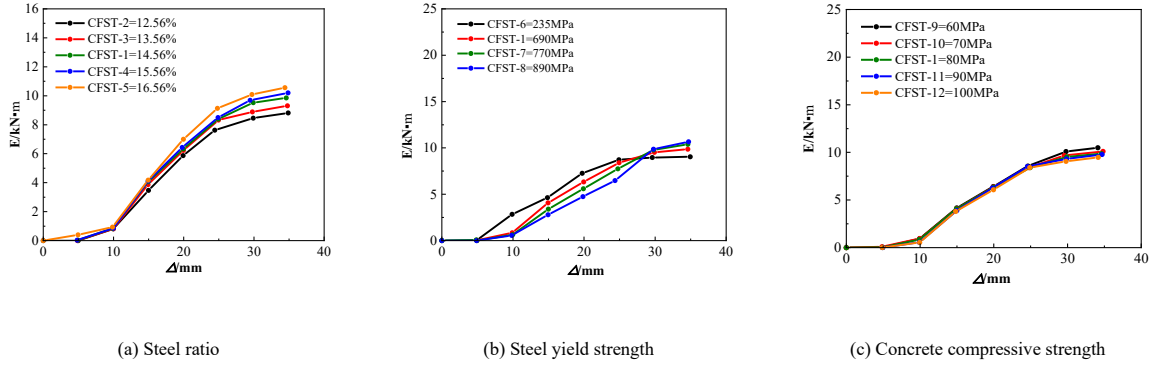


Fig. 11 Energy-dissipation capacity with different parameters

5. Flexural capacity calculation

To determine the flexural bearing capacity of hollow ribbed high-strength concrete filled square steel tubular columns under monotonic loading, this paper adapts and applies the simplified calculation method for the flexural bearing capacity of hollow CFST members as outlined in the T/CCES 7-2020 code[27], along with the calculation method proposed by Professor Han Linhai for the flexural bearing capacity of ribbed concrete filled square steel tubular columns [2]. The modified calculation equations for the flexural bearing capacity of the composite columns in this study is as follows:

$$\frac{1}{\varphi} \frac{N}{N_u} + \frac{a}{b} \frac{M}{M_u} = 1 \quad \left(\frac{N}{N_u} \geq 2\varphi^3 \eta_0 \right) \quad (4)$$

$$-b \left(\frac{N}{N_u} \right)^2 - c \frac{N}{N_u} + \frac{1}{d} \frac{M}{M_u} = 1 \quad \left(\frac{N}{N_u} < 2\varphi^3 \eta_0 \right) \quad (5)$$

$$N_{osc,u} = f_{osc} (A_{so} + A_c) \quad (6)$$

$$f_{osc} = C_1 \chi^2 f_{yo} + C_2 (1.14 + 1.02\xi) f_{ck} \quad (7)$$

$$N_{i,u} = f_{yi} A_{si} \quad (8)$$

In which, N_u and M_u are axial compressive and bending capacity of the member. $N_{osc,u}$ and $N_{i,u}$ are strength of outer and inner steel tube cross sections. A_c , A_{si} and A_{so} are cross section area of concrete, inner and outer steel tube. C_1 and C_2 are steel ratio coefficients. ξ is the confinement coefficient of sandwich concrete. f_{ck} , f_{yi} and f_{yo} are strength of sandwich concrete, inner and outer steel. The indexes a , b , c , d and φ are determined according to references [2] and [27]. η_0 is determined according to Eq. (9)-(10). The bending capacity M_u is determined according to Eq. (11).

$$\eta_0 = (0.5 - 0.245\xi) \cdot (1 + 0.7\chi - 1.8\chi^2) \quad (\xi \leq 0.4) \quad (9)$$

$$\eta_0 = (0.1 + 0.14\xi^{-0.84}) \cdot (1 + 0.7\chi - 1.8\chi^2) \quad (\xi > 0.4) \quad (10)$$

$$M_u = \gamma_{m1} W_{scm} f_{osc} + \gamma_{m2} W_{si} f_{yi} + W_l f_y \quad (11)$$

$$\gamma_{m1} = 1.1 + 0.48 \ln(\xi + 0.1) (1 + 0.06\chi - 0.85\chi^2) \quad (12)$$

$$\gamma_{m2} = 1.04 \chi^{-0.67} - 0.02 \chi^{-2.76} \ln \xi \quad (13)$$

In which, W_{scm} and W_{si} are the section flexural modulus of outer steel tube and inner steel tube and concrete respectively. f_y is the yield strength of ribs. W_l is the section resistance moment of the stiffened plate.

Utilizing the aforementioned calculation method, the flexural bearing capacity of hollow ribbed high-strength concrete filled square steel tubular columns under monotonic loading were computed in this paper. The calculated flexural bearing capacity of the members were then compared with the peak loads obtained from the simulated skeleton curves, as detailed in Table 2 and illustrated in Fig. 12. The error values for the specimens all fall below 10%, with an average error of 6.7% and a variance of 2.1%. This indicates that the discrepancy between the calculated values and the simulated values is relatively small, suggesting that this calculation formula is suitable for determining the flexural bearing capacity of hollow ribbed high-strength concrete filled square steel tubular columns.

Table 2
Comparison of calculated and simulated values

Model No.	$P_{max,s}$ (kN)	$P_{max,c}$ (kN)	Error(%)
CFST-1	434.2	405.2	6.7
CFST-2	324.9	340.6	4.8
CFST-3	389.4	368.1	5.5
CFST-4	460.4	435.6	5.4
CFST-5	476.6	451.3	5.3
CFST-6	390.3	420.1	7.6
CFST-7	468.9	441.5	5.8
CFST-8	495.2	452.8	8.6
CFST-9	415.1	377.3	9.1
CFST-10	429.2	392.3	8.6
CFST-11	445.6	421.1	5.5
CFST-12	448.6	416.9	7.1
Average	-	-	6.7
Variance	-	-	2.1

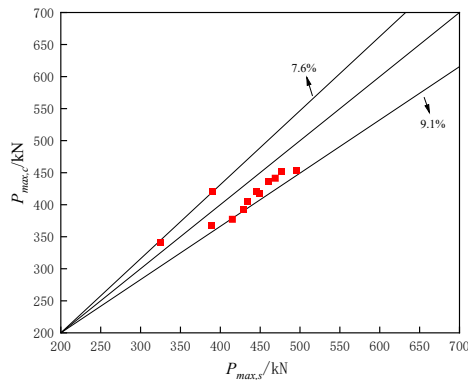


Fig. 12 Comparison of calculated and simulated flexural capacity

6. Conclusions

In this paper, a novel hollow ribbed high-strength concrete filled square steel tubular column is introduced, and the working mechanism as well as the function of the stiffening rib are elucidated. The study investigates the impact of variations in the height of the stiffening rib, the yield strength of the steel tube, and the concrete strength parameters on the hysteretic performance of the member, offering design recommendations. Equations for calculating the flexural bearing capacity suitable for this member is also proposed. The specific conclusions are as follows:

(1) The load-displacement behavior of hollow ribbed high-strength concrete filled square steel tubular columns can be categorized into 3 distinct stages: the elastic stage, the elastic-plastic stage, and the decline stage. During the elastic stage, the load is primarily borne by the steel tube. In the elastic-plastic stage, the steel tube's constraint on the concrete significantly intensifies. In the decline stage, the internal rib plate becomes active, enhancing the member's ductility, delaying the steel tube's bulging, and further reinforcing the steel tube's constraint on the concrete.

(2) Considering parameters such as bearing capacity, stiffness degradation, and energy dissipation capacity, it is advisable to design the member with a stiffening rib height that is within 15% of the section width. The yield strength of the steel tube is recommended to be in the range of 460MPa to 770MPa. Additionally, it is suggested to use concrete with a grade below C90, as higher-grade concrete does not significantly enhance the bearing capacity and may decrease ductility.

(3) In this paper, the proposed calculation method for the flexural bearing capacity of hollow ribbed high-strength concrete filled square steel tubular columns is utilized to determine the flexural bearing capacity of the composite columns. The calculation results show a high degree of consistency, with an average error of 6.7% and a mean square error of 2.1%.

Funding

The research was funded by the Anhui Provincial Natural Science Foundation(2022AHO52286), Open Research Fund of Anhui Provincial Key Laboratory of Building Earthquake Disaster Mitigation and Green Operations(2022-JKYL-001).

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RESEARCH ON THE EXTERNAL RESTRAINED UNITS STIFFNESS OF SEGMENTED ASSEMBLED BUCKLING-RESTRAINED BRACES

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ABSTRACT

The peripheral units of the novel segmented assembled buckling-restrained braces (SABRBs) are assembled by connecting the upper and lower restrained cover plates to the lateral C-shaped steel restraint units using multiple sets of bolts. Owing to the discontinuity of the lateral restraint units, calculating the restraint stiffness becomes relatively complex. In comparison with integrally restrained BRBs, SABRBs necessitate additional consideration of the effects arising from bolt installation. Given the non-uniform lateral forces transferred from the inner core plate to the C-shaped steel restrained units, the SABRBs are assumed to rotate about their width axis. The reduction in stiffness of the peripheral restrained units is primarily attributed to torsional deformation of the plates adjacent to the bolts. In this study, a mechanical model is established that incorporates both the flexibility of bolted connections and the effective area of the restrained plates. By analyzing the deformation mechanism of the peripheral restrained units in the segmented assembled SABRBs, the corresponding force transfer process is examined. Through theoretical derivation and numerical analysis, a formula for the stiffness reduction coefficient μ_0 is derived. Strategies such as optimizing segment layout and adding stiffeners to laterally restrained units are proposed to control this coefficient. Finally, a calculation method for the stiffness of peripheral restrained units is presented. The proposed formula is validated using parametric finite element analyses and experimental data.

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ARTICLE HISTORY

Received: 10 September 2025
Revised: 17 January 2026
Accepted: 22 January 2026

KEYWORDS

Deformation mechanism;
Assembled buckling-restrained
brace;
Restrained Stiffness

1. Introduction

BRBs are highly efficient energy-dissipating and seismic-mitigation units [1-5]. They mainly consist of an axial force-resisting core plate, an outer restrained unit that prevents the core from buckling, and an intermediate unbonded material layer. In recent years, with the rapid development of high-rise and super high-rise buildings both domestically and internationally, assembled BRBs have been increasingly applied in practical engineering. This is due to the advantages of BRBs, such as lightweight construction, ease of installation, replaceable cores after an earthquake, cost-effectiveness and environmental sustainability. Consequently, the types and design theories of BRBs have become more sophisticated [6-13]. Traditional BRBs typically use integrally fabricated restrained units. While this design ensures adequate restrained stiffness, it involves complex manufacturing processes. Furthermore, post-earthquake repairs necessitate replacing the entire BRB assembly, resulting in higher costs and greater operational complexity. Large-tonnage or extra-long BRBs are required for engineering projects, but the use of integral BRBs presents substantial transportation and installation challenges. To address these issues, Zhang Ailin [14] proposed a two-stage assembled BRB, which offers advantages in transportation and installation. Their research confirmed that steel frame-BRB systems incorporating two-stage assembled BRBs exhibit excellent seismic performance. Similarly, Li Guochang [15] developed a novel assembled BRB featuring full-angle steel-restrained units and an aluminum alloy core, demonstrating that it delivers favorable energy dissipation and fatigue resistance while retaining economic and environmental benefits. This study focuses on a segmented assembled BRB [16], as illustrated in Fig. 1. Its external Restrained Units comprise multiple units connected by bolts. Compared with integral BRBs, this configuration significantly simplifies fabrication, transportation, and installation. Additionally, the assembled buckling-restrained brace (BRB) is composed of restrained cover plates, restrained side plates, and a core plate, which are rigidly connected by bolts. These components collectively form an enclosed cavity slightly larger than the core plate—a design intended to allow the core plate ample deformation space under loading. This ensures the core plate can fully harness its energy-dissipating capability, thereby enhancing the structural seismic performance. A notable advantage of the assembled BRB emerges during post-earthquake building repair: only the core plate needs individual replacement, while the external restrained units can be reused intact [17].

The force transmission mechanism of this new segmented assembled BRB differs markedly from that of the conventional integral BRB. The external Restrained Units are bolt-connected and exhibit longitudinal discontinuity, leading to a non-uniform longitudinal stiffness distribution. Accurate assessment of the stiffness of the external Restrained Units is therefore crucial for the structural design of SABRBs.

A new mechanical model is developed that explicitly represents the deformation mechanism of bolt-connected restraining units. The model introduces the concept of effective plate width to quantify the participatory area of cover and side plates in load transfer, and formulates the stiffness reduction via newly derived coefficients. These coefficient captures the combined effects of bolt spacing, bolt cross-sectional position, and the torsional deformation of plates adjacent to bolts—factors previously unquantified in integral BRB design. Key parameters in the theoretical model, namely the effective width coefficients and the stiffness reduction coefficient are determined through extensive parametric finite element analysis. Empirical relationships are established via curve fitting, expressing these coefficients as functions of bolt spacing and the geometric parameters. These fitted equations enable practical design application without requiring complex numerical simulations for each case. The proposed methodology is intended for the design and stiffness evaluation of SABRBs with bolted C-shaped steel restraining units and straight core plates. It applies to braces under elastic behavior of restraining units and does not account for material yielding or bolt preload effects. The method is intended for assembled segmented BRBs with bolted connections.

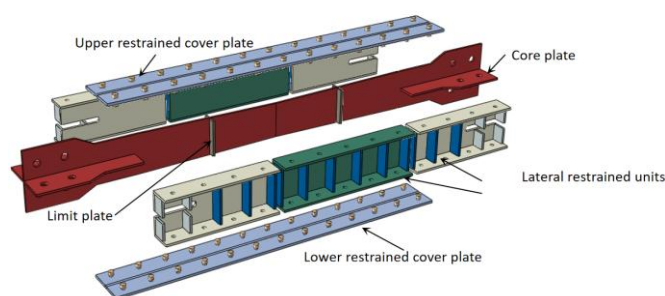


Fig. 1 Disassembly schematic of SABRB

Fig. 2 shows a schematic cross-sectional view of the segmented assembled buckling-restrained brace (BRB), with several key geometric parameters clearly labeled.

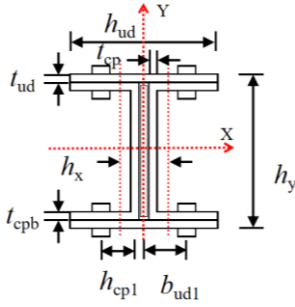


Fig. 2 Sectional schematic of SABRB

In Fig. 2, the width and thickness of the upper (and lower) restrained cover plates are denoted as h_{ud} and t_{ud} , respectively. The height and thickness of the left (and right) lateral restrained units are denoted as h_{cp} and t_{cp} , while the flange width and flange thickness of the left (and right) restrained units are b_{cp} and t_{cpb} , respectively. The distance from the bolt center to the neutral axis of the flange is defined as h_{cp1} , and the distance from the bolt center to the mid-plane of the upper (and lower) restrained cover plates is b_{ud1} . The distance between the neutral axes of the left and right restrained units is denoted as h_x , and the overall height of the external Restrained Units is represented by h_y .

The stiffness performance of the external Restrained Units is primarily reflected in their deformation under the lateral compressive force exerted by the core plate. In this study, the factors influencing the restrained stiffness are investigated by analyzing the deformation behavior of the restrained units.

2. Analysis of factors influencing the stiffness of the external restrained units

2.1. Simplified model

Under axial compression, the SABRB exhibits two-point contact between the core plate and the external Restrained Units at its ends. The lateral compressive force acting at the mid-span can be approximated as a sinusoidally distributed load [18–19]. Thus, the external Restrained Units can be modeled as a simply supported beam subjected to concentrated loads at both ends and a sinusoidally distributed load at the mid-span.

Based on the simplified analysis methods for assembled buckling-restrained braces (BRBs) presented in References 20 and 23, this paper idealizes and simplifies the external restraint units into a three-layer truss system by considering their composition and force transmission characteristics. A half-span of the original system is extracted as the equivalent simplified model: one end, corresponding to the mid-span of the actual structure, is defined as a fixed end; the other end, representing the structural end, is defined as a free end. The equivalent simplified model is illustrated in Fig. 3.

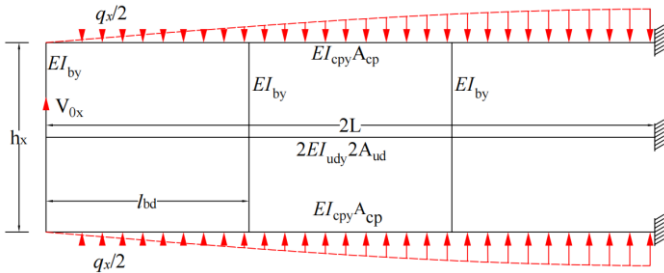


Fig. 3 Equivalent simplified model

The upper and lower chords of the equivalent simplified model represent the left and right lateral restrained units, each with a bending stiffness of EI_{cp} and a cross-sectional area of A_{cp} . The middle chord represents the upper and lower restrained cover plates, with a combined bending stiffness of $2EI_{ud}$ and a total cross-sectional area of $2A_{ud}$. Specific units of the external Restrained Units, together with the bolts, are modeled as web members in the truss system. The spacing between web members corresponds to the distance between adjacent bolts, denoted as l_{bd} , and their bending stiffness—denoted as EI_{by} —is to be determined.

2.2. Influencing factors

For this type of SABRB, if all units are tightly connected, they can theoretically be assumed to function as a fully composite system. However, due to imperfect connectivity between units, the overall bending stiffness of the external Restrained Units is partially reduced. For the three-layer truss simplified model, equilibrium equations can be formulated using the force method to analyze lateral deformation under arbitrary loading, from which the equivalent stiffness of the external Restrained Units can be derived. When a buckling-restrained brace (BRB) is subjected to compressive loading, the deformation mode of its core plate transitions gradually from a single-wave pattern to a multi-wave pattern, as illustrated in Fig. 4. This study focuses primarily on the stiffness variation of the core plate during its single-wave deformation stage. Guo Yanlin [20] developed a differential equation-based model for BRBs with four-channel steel composite restrained units and derived a simplified expression for the restrained stiffness. The key to this method lies in calculating the connection stiffness between restrained units. This analytical approach is also applicable to the SABRB with a straight core plate discussed in this paper.

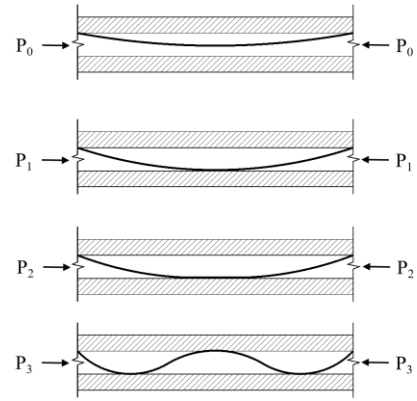


Fig. 4 Variation of core plate buckling modes

When the structure bends about the Y-axis, the sinusoidal lateral compressive force q_x exerted by the core plate on the external Restrained Units is calculated using the following expression [21–22]:

$$q_x = \pi^2 N (v_0 + g_x) \sin(\pi z / L) / L (1 - N / N_{crx}) = \frac{\pi^2}{L^2} C_{qx} \sin(\pi z / L) \quad (1)$$

The concentrated lateral force V_{0x} exerted at the end of the units is calculated as follows:

$$V_{0x} = \frac{1}{2} \int_0^L q_x dz = \pi N (v_0 + g_x) / L (1 - N / N_{crx}) = C_{qx} \pi / L \quad (2)$$

Where, L is the length of the unit; v_0 is the initial imperfection value; N_{crx} is the overall buckling load of the inner core plate when bending about the x-axis; N is the axial force of the inner core plate; g_x is the unilateral gap value along the x-direction between the inner core plate and the peripheral restrained units; and C_{qx} is the maximum mid-span bending moment obtained from the second-order elastic analysis.

By establishing equilibrium equations based on the equivalent simplified model, the maximum lateral deformation under the action of lateral forces can be obtained. By comparing this with the maximum lateral deformation of an idealized unit with uniform stiffness subjected to the same lateral load, the equivalent lateral restrained stiffness of the external Restrained Units can be derived. If the stiffness of the web members is known, a simplified calculation method for the restrained stiffness can be derived using a modified version of the "continuous strut-tie model"—a model commonly applied to multi-branched reinforced concrete shear walls. In this approach, the web connections between the upper, middle, and lower chords are idealized as continuous elements [23].

When the units rotate about the Y-axis, two parameters are defined as follows:

$$\gamma_1 = 6I_{2y}L^2 / (I_{cp} + I_{ud})l_{bd}h_x \quad (3)$$

$$\gamma = 6I_{2y}L^2 / (I_{cp} + I_{ud})I_{bd}h_x + 24I_{2y}L^2 / A_{cp}I_{bd}h_x^3 \quad (4)$$

Let:

$$\lambda(\gamma, \gamma_1) = 4\gamma_1^2 / 4\gamma^2 + \pi^2 \quad (5)$$

Then, the stiffness reduction coefficient of the external Restrained Units is expressed as:

$$\mu_y = 1 \times 2(I_{ud} + I_{cp}) / [I_y \times 1 - \lambda(\gamma, \gamma_1)] \quad (6)$$

Where, I_y is the moment of inertia of the external Restrained Units as a whole about the Y-axis:

$$I_y = 2(I_{ud} + I_{cp}) + A_{cp}h_x^2 / 2 \quad (7)$$

3. Theoretical analysis of restrained coupling stiffness

3.1. Force analysis

The coupling stiffness of the external Restrained Units is jointly provided by the upper and lower restrained cover plates, the left and right lateral restrained units, and the bolts. It primarily reflects the unit's capacity to resist relative longitudinal sliding deformation between units. To investigate this, models with varying longitudinal dimensions of the restrained units were developed for analysis. Since the stiffening ribs on the lateral restrained units have a negligible influence on relative sliding deformation, and the segmentation of the lateral restrained units complicates stiffness evaluation, this section disregards the effects of stiffener placement and segmentation of the lateral restrained units. When the entire restrained unit bends about the Y-axis, a pair of equal and opposite forces is hypothetically applied at the neutral axis of the left and right lateral restrained units. This pair of unbalanced forces is transmitted from point A_1 on the left restrained to point B_1 on the right restrained

via the following load path (see Fig. 5): starting from A_1 , the force is transferred through the left flange, then via the bolts to the upper (or lower) restrained cover plate, then along the cover plate between bolts, through another bolt, and finally to the right lateral restrained and its flange, reaching B_1 . The plates and bolts along this transmission path are idealized as web members, and their deformation directly influences the coupling stiffness of the restrained system.

The coupling stiffness can be further divided into two components: One part is provided by the in-plane stiffness of the restrained plates along the force transmission path. The other part stems from the out-of-plane bending stiffness, which is attributed to bolt rotation and longitudinal tilting during force transfer—this induces local out-of-plane deformation in the adjacent restrained plates.

3.2. Influence of in-plane plate stiffness on coupling stiffness

In the deformation calculation of the external Restrained Units, not all plates contribute to stiffness due to the discrete distribution of bolts. A numerical model of the buckling-restrained brace was established via ABAQUS, with an initial force applied about the Y-axis. In this study, the identical geometric and material parameters to those of the test specimen were adopted for model construction. The finite element model (FEM) accurately reproduced the geometric configuration of the bolts. In ABAQUS finite element analysis, the selection of solid element type and mesh size exerts a significant influence on the accuracy, computational efficiency, and stability of the model results. By reviewing the commonly used ABAQUS solid element types reported in relevant literature, this study compiled the findings in Table 1. Considering that the model involved in this paper requires extensive simulation of contact interactions, the C3D8R element is deemed the more suitable choice. The element size of the model is about 15 mm, as shown in Fig. 5(b). As revealed by the force analysis, during force transmission, only the bolts function as connecting units at the joint locations of the restrained units, while the plates immediately adjacent to the bolts do not provide any stiffness. However, for plates farther from the bolted joints, the area of the plates involved in load-bearing gradually increases; such regions are referred to as "effective plates." The effective zones of the upper/lower and left/right restrained units are illustrated in Fig. 5(a).

Table 1

Comparisons of different 3D element type

Type	Advantages	Disadvantages
C3D8: An 8-node linear brick.	Suitable for regular-shaped elements, enabling precise integration of polynomials within the stiffness matrix.	Risk of shear locking under bending loads, resulting in overestimated stiffness calculations.
C3D8R: An 8-node linear brick, reduced integration, hourglass control.	One fewer integration point per direction than fully integral elements, offering computational efficiency and superior adaptability to bending problems with reduced susceptibility to shear locking.	Sensitive to hourglass, potentially yielding excessively soft results.
C3D8I: An 8-node linear brick, incompatible modes.	Overcomes shear locking by introducing additional degrees of freedom, achieves accuracy close to quadratic elements, and has lower computational cost.	Sensitive to element distortion; unsuitable for contact analysis or large deformation problems.

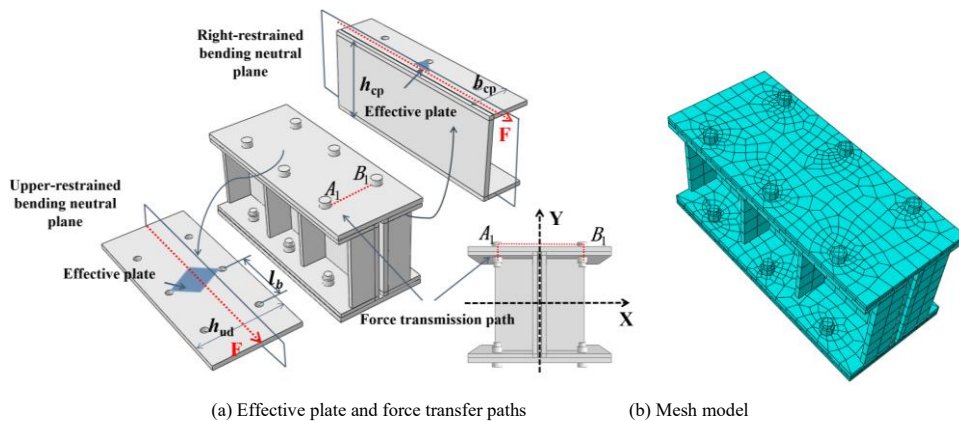


Fig. 5 Analysis model of SABRB

For the effective plate elements on the left (or right) restrained unit, the distance from the bolt center to the bending neutral axis of the left restrained unit is denoted as h_{cp} . It should be noted that within the range of the effective region, the distribution of the effective area is complex and difficult to define precisely. Accordingly, appropriate simplifications are employed in the subsequent calculations. The effective flange plate of the left restrained units is

replaced by an equivalent strip with a uniform width, as illustrated in Fig. 5. This width is defined as the equivalent width, and its value is expressed as:

$$h_{cpy} = \alpha_{cp} \times h_{cp} \quad (8)$$

Where, α_{cp} is the effective width coefficient for the effective plate area on the left (or right) restrained unit.

Based on the force transmission path illustrated in Fig. 6, the load-transfer model can be simplified as a beam with one end pinned and the other end fixed. The pinned end corresponds to the bolted connection (i.e., the midpoint of the bolt shank), while the fixed end corresponds to the bending neutral axis of the plate, as illustrated in Fig. 5. The moment of inertia of this unit is:

$$I_{cp} = h_{cp}^3 t_{cpb} / 12 \quad (9)$$

Based on the simplified beam model, the longitudinal deformation Δ_{cp} induced by the longitudinal force F in the left (or right) restrained unit is expressed as [24]:

$$\Delta_{cp} = F h_{cp}^3 / 3EI_{cp} \quad (10)$$

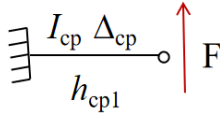


Fig. 6 Simplified force analysis of effective plates in left and right restrained units

The effective plate elements on the upper (or lower) restrained unit are analyzed using a similar approach. For the upper restrained unit, the distance from the bolt center to the bending neutral axis is denoted as b_{ud1} . By adopting the same modeling method applied to the left (or right) restrained unit, a simplified beam model is established, as illustrated in Fig. 7. In this model, the pinned end corresponds to the bolted connection, while the fixed end corresponds to the bending neutral axis of the upper restrained plate. The equivalent width and moment of inertia of the effective plate section are expressed as:

$$h_{udy} = \alpha_{ud} h_{ud} / 2 \quad (11)$$

$$I_{ud} = h_{udy}^3 b_{ud1} / 12 \quad (12)$$

Where, α_{ud} is the effective width coefficient for the upper (or lower) restrained unit.

According to this simplified model, the longitudinal deformation Δ_{ud} induced by the longitudinal force F in the upper (or lower) restrained unit is expressed as:

$$\Delta_{ud} = F h_{ud1}^3 / 3EI_{ud} \quad (13)$$

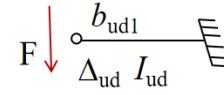


Fig. 7 Simplified force analysis of effective plates in upper and lower restrained units

Thus, the total in-plane deformation of the restrained plates is:

$$\Delta = \Delta_{ud} + \Delta_{cp} = F h_{cp1}^3 / 3EI_{cp} + F h_{ud1}^3 / 3EI_{ud} = F \delta_y \quad (14)$$

Where, δ_y is the coupling flexibility coefficient.

Through qualitative analysis of the effective plate area, it is found that the effective width coefficients α_{cp} and α_{ud} are influenced by the longitudinal bolt spacing l_{bd} and the distance from the bolt center to the bending neutral axis of the upper/lower and left/right restrained units. When the longitudinal bolt spacing decreases, the effective plate regions adjacent to neighboring bolts begin to overlap, thereby reducing the total effective area and resulting in a decrease in the effective width coefficient. Conversely, when the bolt spacing becomes sufficiently large, the effective width coefficient is no longer sensitive to changes in spacing. Similarly, when the distance from the bolt center to the bending neutral axis decreases, the spread of the effective plate area is limited, which also leads to a reduction in the effective width coefficient.

Table 2
Model parameters

Model	h_{cp}	t_{cp}	b_{cp}	t_{cpb}	h_{ud}	t_{ud}	h_{cp1}	b_{ud1}	l_{bd}
Model 1A	224	11.5	100	10	214	12	40	45	50-200
Model 1B	224	11.5	100	10	214	12	56	61	50-200
Model 1C	224	11.5	100	10	214	12	30-60	35-65	100
Model 1D	224	11.5	100	10	214	12	30-60	35-65	150
Model 2A	284	11.5	170	10	350	12	51	56	85-306
Model 2B	284	11.5	170	10	350	12	85	91	85-306
Model 2C	284	11.5	170	10	350	12	51-102	56-107	140
Model 2D	284	11.5	170	10	350	12	51-102	56-107	255
Model 3A	224	11.5	100	10	214	12	37.5	42.5	53.5-214
Model 3B	224	11.5	100	10	214	12	51	56	53.5-214
Model 3C	224	11.5	100	10	214	12	27.1-69.9	32.1-74.9	107
Model 3D	224	11.5	100	10	214	12	27.1-69.9	32.1-74.9	160.5
Model 4A	284	11.5	170	10	350	12	65	70	87.5-315
Model 4B	284	11.5	170	10	350	12	100	105	87.5-315
Model 4C	284	11.5	170	10	350	12	47.5-117	52.5-122	140
Model 4D	284	11.5	170	10	350	12	47.5-117	52.5-122	262.5
Model 5A	304	11.5	180	10	370	12	72	77	144-270
Model 5B	304	11.5	180	10	370	12	108	115	144-270
Model 5C	304	11.5	180	10	370	12	45-126	50-131	108
Model 5D	304	11.5	180	10	370	12	45-126	50-131	144
Model 5E	304	11.5	180	10	370	12	90	95	144-270

This study adopts a semi-analytical method combined with finite element analysis (FEA) to solve the effective width coefficient. Several configurations of external Restrained Units are designed by varying the bolt spacing (l_{bd}) and the bolt cross-sectional position (b_{ud1}). Corresponding finite element models are

developed, with cross-sectional parameters summarized in the relevant Table 2. Contact relationships between the upper, lower, left, and right restrained units are defined. Assuming n groups of bolts are arranged longitudinally, a pair of opposing forces, F_1 , is applied to the upper and lower restrained cover plates.

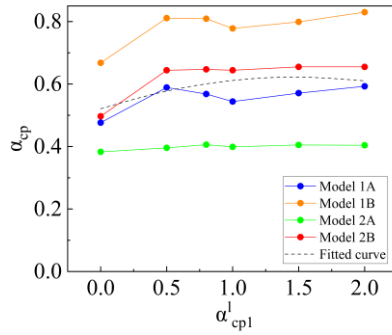
The force acting on each bolt group is thus $F=F_1/n$. Finite element analysis yields the relative displacement Δ_x between the upper and lower neutral axes. The flexibility coefficient of the bolted connection can therefore be expressed as: $\delta_x=\Delta_x/F$. This coefficient is then used to evaluate the corresponding stiffness and effective width coefficients. To determine the effective width coefficient α_{cp} for the left and right restrained units, a pair of equal and opposite longitudinal forces is applied along the cut edges at the bending neutral axis of the left and right restrained units. The resulting longitudinal displacement of nodes on these edges is recorded. To ensure that the computed displacement exclusively represents the deformation of the lateral restraints, the upper and lower restrained cover plates are modeled as rigid bodies.

3.2.1. Determination of the coefficient α_{cp} .

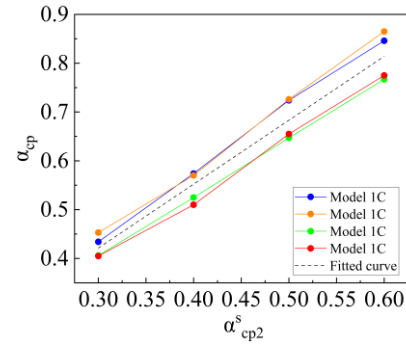
Two coefficients, α_{cp1} and α_{cp2} , are defined to respectively represent the effects of longitudinal bolt spacing l_{bd} and the bolt's cross-sectional position h_{cp1} , and they are expressed as:

$$\alpha_{cp1} = l_{bd} / b_{cp} \quad (15)$$

$$\alpha_{cp2} = h_{cp1} / b_{cp} \quad (16)$$



(a) Varying the bolt spacing



(b) Varying the bolt cross-sectional position

Fig. 8 Fitting calculation of the coefficient α_{cp}

3.2.2. Determination of the coefficient α_{ud}

Similarly, two coefficients, α_{ud1} and α_{ud2} , are defined to account for the effects of the bolt spacing l_{bd} and the bolt's cross-sectional position b_{ud1} , respectively, and are expressed as:

$$\alpha_{ud1} = 2l_{bd} / h_{ud} \quad (19)$$

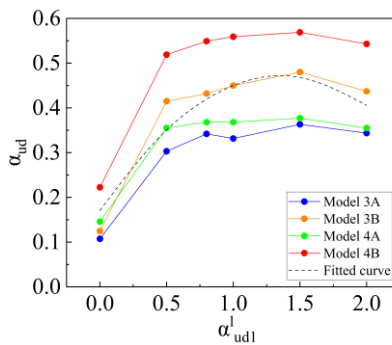
$$\alpha_{ud2} = 2b_{ud1} / h_{ud} \quad (20)$$

The relationships between these two parameters and the coefficient α_{ud} are illustrated in Fig. 8. The fitted equations for the parameters and the coefficient

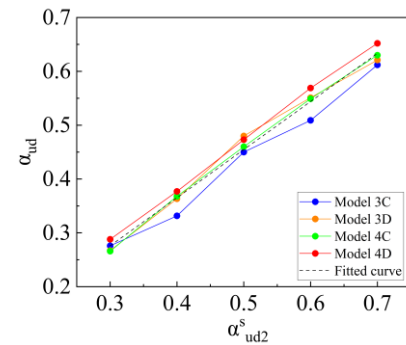
α_{ud} are given as follows:

$$\alpha_{ud} = \left\{ -0.1616(\alpha_{ud1})^2 + 0.4407\alpha_{ud1} + 0.1707 \right\} (0.8935\alpha_{ud2} + 0.0079) \quad (21)$$

Where, α_{ud1} denotes the influence factor associated with the longitudinal bolt spacing l_{bd} , and α_{ud2} represents the influence factor related to the bolt's cross-sectional position b_{ud1} .



(a) Varying the bolt spacing



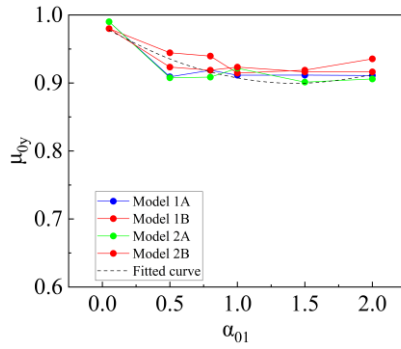
(b) Varying the bolt cross-sectional position

Fig. 9 Fitting calculation of the coefficient α_{ud}

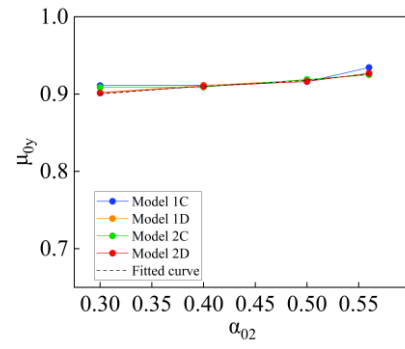
3.3. Coupling stiffness associated with the out-of-plane stiffness of restrained plates

The upper and lower restrained cover plates are connected to the left and right restrained units via bolts. Due to bolt deformation, rotational displacement occurs between the mid-planes of these units, accompanied by longitudinal displacement; these combined effects result in a reduction in the overall stiffness of the external Restrained Units. Given the complex interaction between the connected plates, a correction factor is introduced to the previously derived stiffness of the external Restrained Units to account for the influence of rotational deformation. This correction further refines the analysis of factors affecting the stiffness of the SABRB external Restrained Units.

To incorporate the effects of rotational deformation, the finite element model is enhanced by introducing out-of-plane restraints between the upper/lower cover plates and the flanges of the left/right restrained units. By comparing the relative sliding deformation Δ obtained from this improved model with the Δ derived in Section 2.2, the stiffness reduction coefficient due to rotational deformation, denoted as μ_{0y} , can be determined.



(a) Varying the bolt spacing



(b) Varying the bolt cross-sectional position

Fig. 10 Calculation of the reduction coefficient μ_{0y}

3.4. Determination of restrained stiffness

Considering the influence of out-of-plane deformation of the restrained plates, Equation (14) should be revised as:

$$\Delta = F\delta_y / \mu_{0y} \quad (24)$$

If the longitudinal deformation is calculated using the simplified model illustrated in Fig. 3, then:

$$\Delta = Fh_x^3 / 48EI_{by} \quad (25)$$

By comparing the two expressions, the expression for EI_{2y} can be derived as:

$$EI_{by} = \frac{0.9(0.0402\alpha_{01}^2 - 0.116\alpha_{01} + 0.9833)h_x^3}{48 \left(\frac{h_{cp1}^3}{3EI_{cp}} + \frac{h_{ud1}^3}{3EI_{ud}} \right)} \quad (26)$$

The stiffness of the restrained units can thus be expressed as $\mu_y EI$, where I is the composite moment of inertia of the external Restrained Units, calculated under the assumption of full composite action based on the plane section hypothesis.

4. Finite element analysis verification of the stiffness reduction coefficient for the external restrained units

4.1. Example verification of the stiffness reduction coefficient

To verify the accuracy of the theoretical formula derived in the previous

Two parameters are defined: α_{01} , which represents the influence of bolt spacing on the reduction coefficient μ_{0y} , and α_{02} , which represents the influence of the bolt's cross-sectional position on μ_{0y} . The effects of these parameters are illustrated in Fig. 10. Based on the data points presented in Fig. 10(a), a fitted expression describing the influence of variations in bolt spacing on the reduction coefficient is derived as:

$$\mu_{0y} = 0.0402\alpha_{01}^2 - 0.116\alpha_{01} + 0.9833 \quad (22)$$

As shown in Fig. 10(b), although the bolt's cross-sectional position influences the reduction coefficient, the variation is relatively small and generally remains around 0.9. Thus, the corresponding fitted expression for the reduction coefficient is:

$$\mu_{0y} = 0.9(0.0402\alpha_{01}^2 - 0.116\alpha_{01} + 0.9833) \quad (23)$$

section, a numerical case study is executed. The Model 5 series listed in Table 2 is chosen for validation. With both ends of the SABRB assumed to be pinned, a sinusoidally distributed load is imposed on the left and right restrained units. The resulting displacements under varying bolt cross-sectional positions and bolt spacings are analyzed and then compared with those of a monolithic BRB under the same loading conditions. From this comparison, the stiffness reduction coefficient of the segmented external Restrained Units is deduced, which is subsequently compared with the theoretical coefficient derived earlier. The displacement of a monolithic BRB under the same loading condition can be calculated using the following formula:

$$\Delta = q_s L^4 / \pi^4 EI \quad (27)$$

Where, I is the composite moment of inertia of the external Restrained Units assuming full composite action, calculated based on the plane section hypothesis, and L is the length of the BRB.

The stiffness reduction coefficients obtained from the simplified calculation formula and finite element analysis (FEA) are both presented in Fig. 11. As illustrated in Fig. 11(a), (c), despite minor discrepancies between the two sets of results, the calculation error does not exceed 10%, and the overall trend is consistent. To more intuitively demonstrate the minimal discrepancy between the results derived from the calculation formulas and those obtained via finite element analysis, a bar chart for result comparison was added to facilitate a more straightforward understanding of the accuracy of the calculation formulas, as illustrated in Fig. 11(b) and 11(d). It is noteworthy that for very short BRB specimens with large bolt spacing, the calculated stiffness reduction coefficient may be significantly low, which does not reflect practical behavior. However, when bolts are arranged with sufficient density, the simplified formula yields conservative results, thereby enhancing safety. Therefore, using the simplified formula to estimate the stiffness of the external Restrained Units in BRBs is considered reasonable and practical.

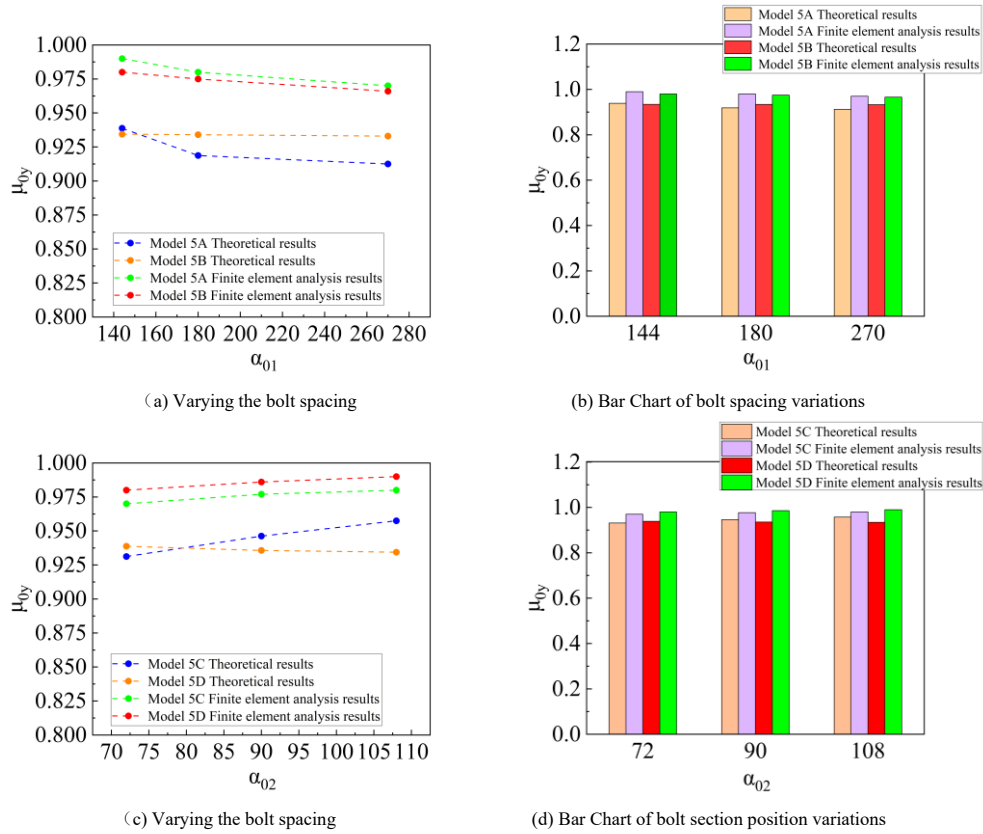


Fig. 11 Comparison between the simplified theoretical formula results of μ_{oy} and the calculation results of the finite element analysis

4.2. Analysis of Factors influencing the stiffness of the external restrained units

In the novel SABRB, the external Restrained Units incorporate transverse stiffening ribs, with the lateral restrained units adopting a segmented configuration. These design features exert a significant influence on the overall stiffness of the external restrained units. To examine this influence, the parameters of Model 5A, 5C, 5D, and 5E are modified to generate Model 5F1~3, 5G1~3, 5H1~3, and 5J1~3, respectively, as summarized in Table 3.

The transverse stiffening ribs are designed with a thickness identical to that of the flanges of the left and right restrained units, and a width equal to half the flange width. In the proposed model, the left and right restrained units are divided into three segments, which are aligned with the positions of the limiting plates on the inner core plate to optimize material utilization. Given that this study focuses primarily on the stiffness of the external Restrained Units, the lengths of these three segments are adjusted accordingly when bolt spacing is varied—this adjustment not only reflects practical restraints but also more accurately captures how segmentation influences stiffness. A sinusoidally distributed load is applied to the left and right restrained units to compute the mid-span displacement of the external Restrained Units. These displacements

are then compared with those from unmodified models to assess the impacts of lateral restrained segmentation and stiffening rib configuration on stiffness. The spacing of the transverse stiffening ribs is set to 0.6 times the height h_{cp} of the SABRB. In numerical simulations, by varying the bolt spacing and cross-sectional position, changes in the displacement of the external Restrained Units are recorded—these changes, as illustrated in Fig. 12, directly reflect variations in restrained stiffness. Finite element results reveal that segmenting the left and right restrained units, coupled with the addition of transverse stiffeners, effectively reduces the mid-span displacement of the SABRB external Restrained Units—an indication of increased stiffness. Specifically, when the rib spacing is set to $0.6h_{cp}$, the displacement is significantly smaller than in the unmodified model. As shown in Fig. 12, variations in bolt spacing and cross-sectional position lead to only minor displacement changes, with correspondingly limited variations in stiffness. For the models in Table 3 that share the same letter but different numbers, the differences arise from the adjustment of bolt parameters or bolt cross-sectional positions. Therefore, these models are not labeled individually in Fig. 12, but are represented by the same corresponding letter.

Table 3

Parameter table for different models of external restrained units

Model	h_{ep}	t_{ep}	b_{ep}	t_{cpb}	h_{ud}	t_{ud}	h_{ep1}	b_{ud1}	l_{bd}
Model 5F1	304	11.5	180	10	370	12	72	77	144
Model 5F2	304	11.5	180	10	370	12	72	77	180
Model 5F3	304	11.5	180	10	370	12	72	77	270
Model 5G1	304	11.5	180	10	370	12	72	77	108
Model 5G2	304	11.5	180	10	370	12	90	95	108
Model 5G3	304	11.5	180	10	370	12	108	113	108
Model 5H1	304	11.5	180	10	370	12	72	77	144
Model 5H2	304	11.5	180	10	370	12	90	95	144
Model 5H3	304	11.5	180	10	370	12	108	113	144
Model 5J1	304	11.5	180	10	370	12	90	95	144
Model 5J2	304	11.5	180	10	370	12	90	95	180
Model 5J3	304	11.5	180	10	370	12	90	95	270

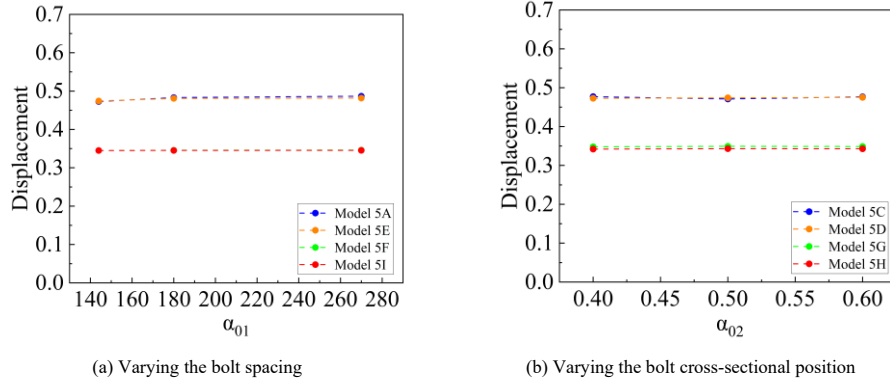


Fig. 12 Influence of discontinuous lateral restrained units and transverse stiffeners on external restrained stiffness

To more distinctly demonstrate the discrepancy between the stiffness calculation formula derived in this paper and the results obtained via the conventional method, the bending stiffness of Model 5A and Model 5E was calculated using both approaches, as shown in Fig. 13, the discrepancy between the flexural rigidity of the buckling-restrained brace calculated by the simplified method and that derived from the theoretical formula proposed in this paper reaches 29.3%.

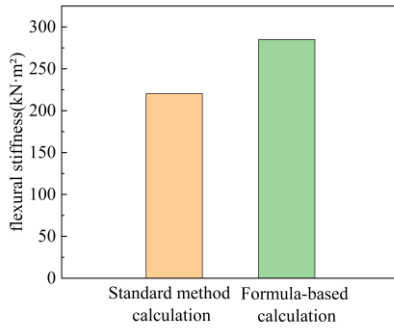


Fig. 13 Comparison of bending stiffness calculated by different methods

Table 4
Correction coefficient

Model	correction coefficient α
5F1	1.37
5F2	1.37
5F3	1.382
5G1	1.374
5G2	1.398
5G3	1.348
5H1	1.382
5H2	1.393
5H3	1.406
5J1	1.367
5J2	1.396
5J3	1.394

The model correction coefficient α is derived by comparing the stiffness of the models in Table 4 with that of the baseline model, and the specific values are presented in Table 4. It is assumed that the values of the correction coefficient α obey a normal distribution, and the lower limit of the 99.5% confidence interval is adopted. In this case, α can be expressed as [25]:

$$\alpha = \alpha_m - t_{0.05}(n_0 - 1)S / \sqrt{n_0} \quad (28)$$

where, α_m is the average of α ; n_0 is the number of specimens; S is the sample standard deviation; $t_{0.05}(n_0 - 1)$ is the value of t distribution. α is calculated according to equation (28) and Table 4, whose value equals to 1.382. Thus, the suggest value of α is simplified to be 1.38.

Consequently, when the left and right restrained units are segmented by practical design requirements and stiffening ribs are arranged at 0.6 times the unit height, the stiffness of the peripheral constrained unit should be amplified by a factor of 1.38. Specifically, the bending stiffness of the buckling-restrained brace calculated by the conventional method is 220.5 kN·m², while that of the segmented assembled buckling-restrained brace calculated by the theoretical formula proposed herein is 285.1 kN·m². The result of the proposed method deviates by 29.3% from that of the conventional method, indicating a significant difference. This verifies that the stiffness calculation formula derived in this study exhibits substantial guiding significance for the design of this type of buckling-restrained brace.

The flow for calculating the external restraining stiffness of this type of buckling-restrained brace using theoretically derived formulas is illustrated in Fig. 14.

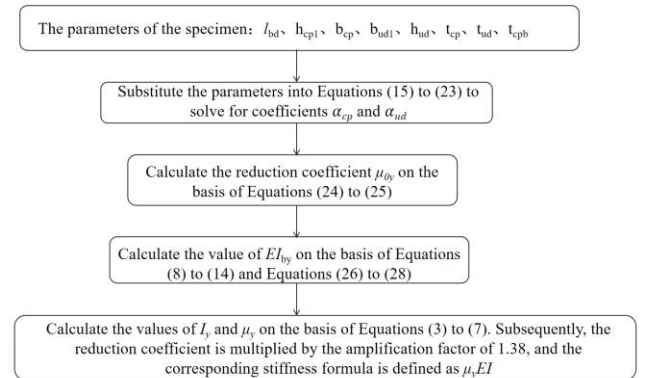


Fig. 14 Calculation process for stiffness of external restraint units of SABRB

5. Experimental validation

5.1. Compression test of external restraining units

To further verify the rationality of the theoretical derivation formulas in this paper, experiments are now carried out for verification. Q235 steel is used to fabricate the specimen of the externally restrained units of the SABRB, as shown in Figs. 15 and 16. Since the actual and simulated force-bearing positions of the external restrained units are internal, to better reflect the actual situation, half of the lateral restrained units are assembled in the experiment. The specimen is placed on the hinged support, and a mass plate is placed inside. The total mass of the mass plate is 30kg. A sinusoidal loading is simulated, and the loading diagram is shown in Fig. 17. A hydraulic device is used for loading, as shown in Fig. 18. To fully exploit the material properties, the loading speed is controlled at 2mm/min throughout the process to obtain the ultimate bearing capacity of the external restrained units. Then, its flexural stiffness is calculated and compared with the restrained stiffness obtained from the theoretical derivation formulas in this paper.

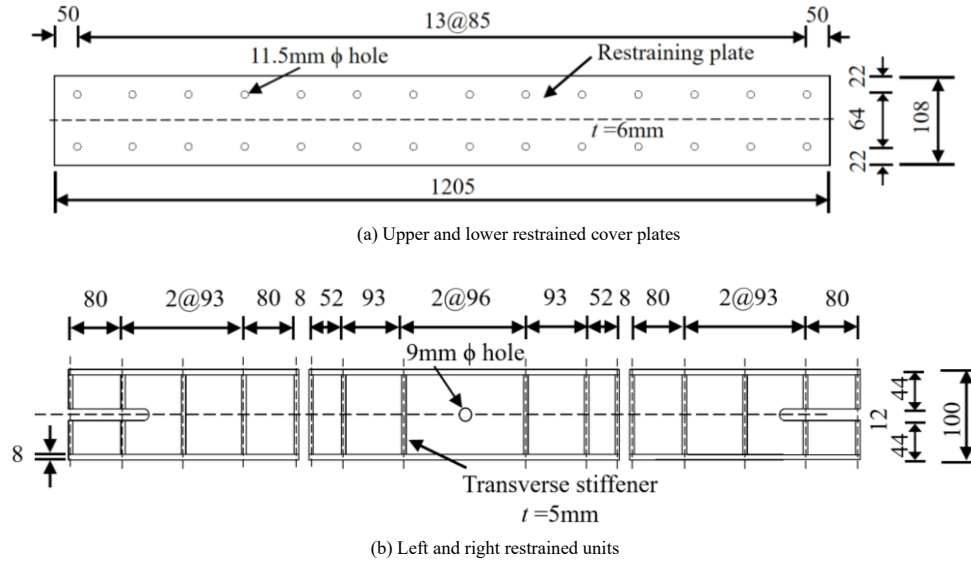


Fig. 15 Dimension of specimen



Fig. 16 Specimen installation

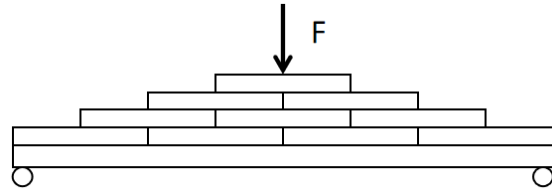


Fig. 17 Schematic diagram of loading

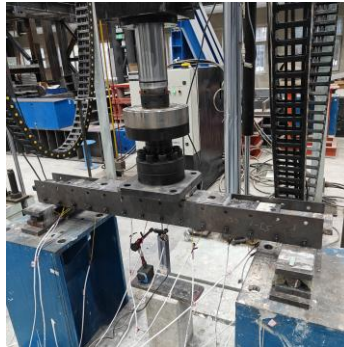


Fig. 18 Hydraulic loading

The load-displacement curve of the external restrained units of the SABRB throughout the entire loading process is shown in Fig. 20(a). The ultimate load in the elastic stage, as illustrated in Fig. 20(b), corresponds to a displacement of 20.01 mm and a bearing capacity of 187.63 kN. Using the theoretical derivation formula in this paper, the flexural stiffness of the externally restrained units is calculated as 165.7 kN·m². The formula for the maximum mid-span deflection is as follows:

$$v = 5ql^4 / 384EI \quad (29)$$

Substituting the values and simplifying, the actual experimental flexural stiffness EI is determined to be 143.3 kN·m². During the test, strain gauges were

evenly attached to the left and right restrained units. To better illustrate the deformation behavior, data from the middle strain gauges N_1 and N_2 are presented as representative, as shown in Fig. 19. From the test data in Fig. 23, it is evident that the strain at location N_1 reached only 570 $\mu\epsilon$, and that at location N_2 reached only 350 $\mu\epsilon$. As the loading displacement increased, the specimen underwent global buckling accompanied by significant local deformation, and the test was terminated immediately, as shown in Figs. 21 \text{ \textasciitilde } 22. The error between the theoretically derived formula and the experimentally obtained flexural stiffness is 14.8%, which is relatively small, indicating a relatively low error level. This indicates that the theoretical framework proposed in this paper aligns well with actual stress conditions, providing robust support for calculating the stiffness of the external Restrained Units of BRBs and laying a theoretical foundation for subsequent research.

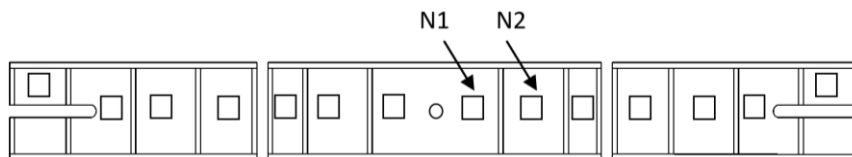
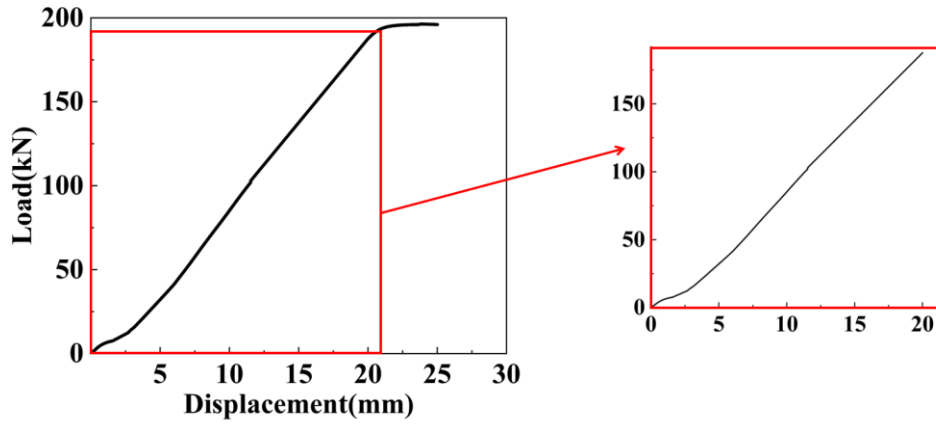


Fig. 19 Strain gauge layout



(a) The full-range load-displacement curve

(b) The elastic-stage load-displacement curve

Fig. 20 The load-displacement curve



Fig. 21 The global buckling

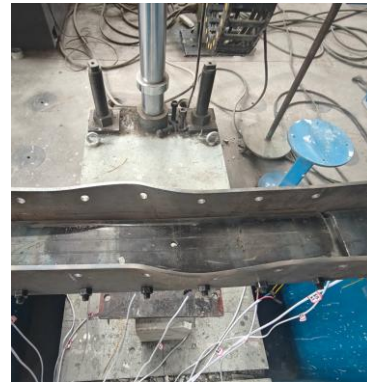


Fig. 22 The local deformation

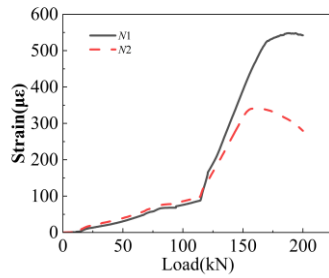


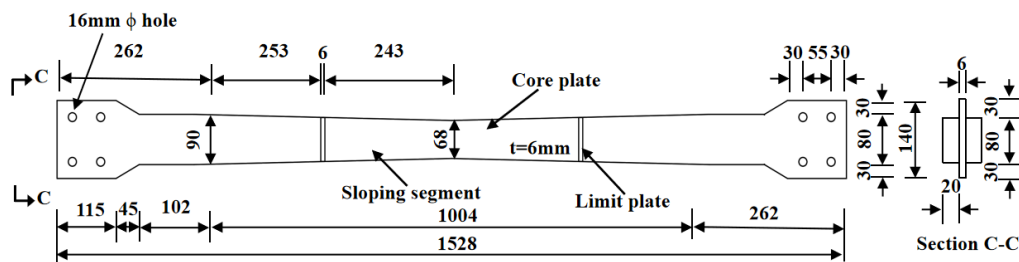
Fig. 23 Strain-load curve

5.2. Overall axial compression test

To further validate the flexural stiffness of the external restraining

components of this assembled buckling-restrained brace, a full-scale axial compression test was performed. The fabrication details of the test specimen are presented in Figure 24. Both the core plate and restrained units were fabricated from Q235 steel.

A servo-hydraulic actuator was adopted as the test loading device to apply cyclic loads to the specimen, which was firmly mounted on a rigid loading frame to ensure the stability of the brace. The SABRB was connected to the floor beam in the structural assembly, and the overall installation configuration is detailed in Fig. 25. The test adopted a tension-first-then-compression loading protocol, and the loading regime of the specimen is presented in the table 5. When the displacement was loaded to 31.2 mm (1/50 of the brace length) and the third loading cycle was initiated, brittle fracture occurred in the region of the core plate end without lateral restraint units. The forward loading was immediately terminated, and an attempt was made to apply reverse compression; however, the specimen had already completely lost its load-bearing capacity, and the test was officially concluded. The external restraint units maintained elastic deformation throughout the entire test process. The final failure mode of the brace is detailed in Fig. 26.



(a) Core plate

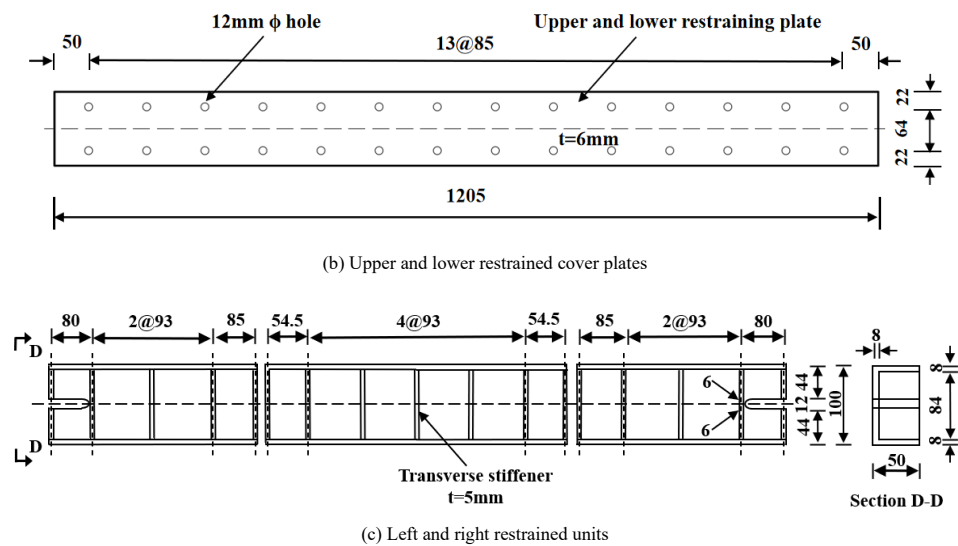


Fig. 24 Machining dimension drawing of units

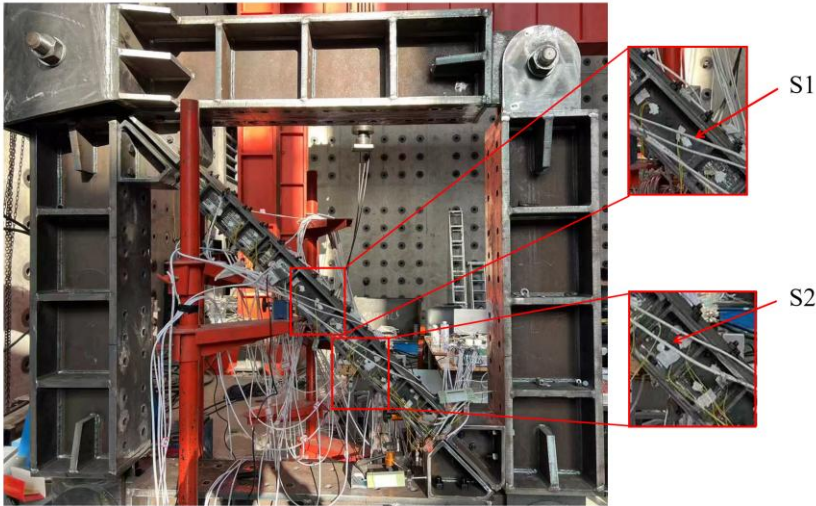


Fig. 25 Test setup of SABRB

Table 5
Loading system

Structural height (mm)	Axial strain	Displacement Amplitude (mm)	Loading Cycles
1560	1/300	6.24	3
	1/200	7.8	3
	1/150	10.4	3
	1/100	15.6	3
	1/50	31.2	3

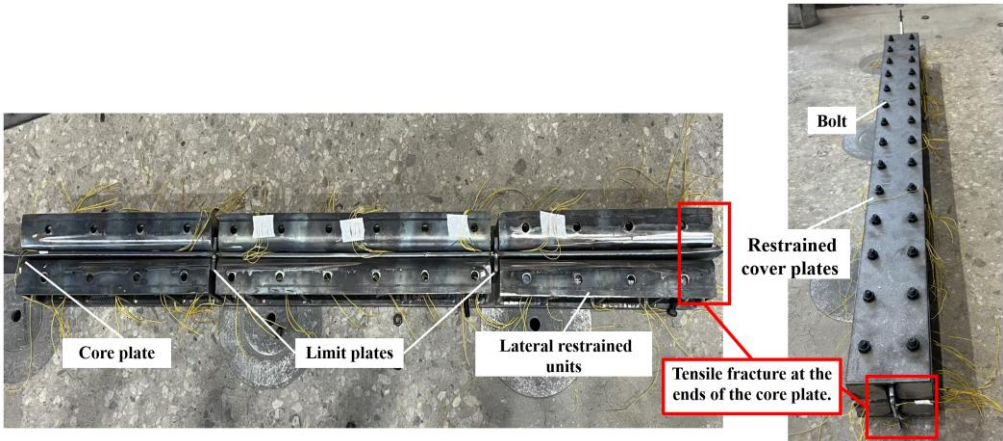


Fig. 26 Specimen failure

To measure the variations in the external restraining components during the test, strain gauges were uniformly affixed to them. For better illustration of their deformation characteristics, S1 and S2 were selected as typical measurement points, as shown in Fig. 25. As can be seen from Fig. 27, the strains of S1 and S2 were obtained. For Q235 steel, its yield strain ε_y was calculated by the formula $\varepsilon_y = f_y / E$, resulting in $\varepsilon_y = 235 / 210000 = 1.119 \times 10^{-3}$, equivalent to 1119

$\mu\epsilon$. Throughout the loading process, the external restrained units remained in an elastic state, with their strains consistently lower than the yield strain. Therefore, it can be concluded that the external restraining components were capable of effectively resisting the lateral pressure exerted by the core plate and restricting the deformation of the core plate during the entire axial loading process, thus ensuring the normal operation of the assembled buckling-restrained brace.

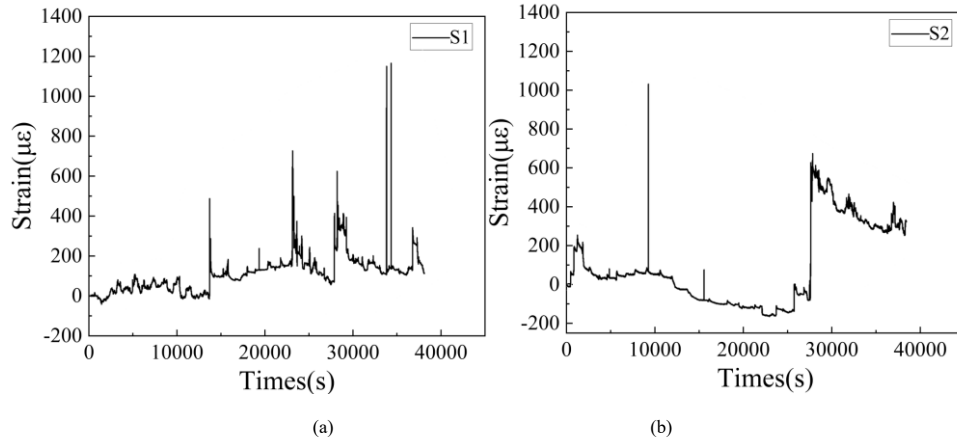


Fig. 27 Strain during loading process

6. Conclusions

This paper investigates the mechanism and stiffness of the external Restrained Units in an SABRB. Since the restrained units are bolt-connected, the load-bearing behavior of the external restrained units in SABRBs differs from that in integral BRBs, with corresponding variations in the stiffness of the external restrained units. Based on a simplified truss model, this paper derives and fits a formula for calculating the stiffness reduction coefficient of the external restrained units through an analysis of its load-bearing mechanism and numerical simulations. The formula incorporates the influences of bolt spacing and bolt cross-sectional position, and its rationality is verified via numerical analyses. Additionally, the segmentation of left and right restrained units and the arrangement of transverse stiffeners can also induce changes in the stiffness

of the external restrained units. Through comparisons of numerical simulation data, a reasonable layout scheme for transverse stiffeners is determined: when transverse stiffeners are arranged at 0.6 times the height of the external restrained units, the stiffness of the external restrained units in SABRBs can match that of integral BRBs. This study provides a stiffness reduction coefficient, proposes a formula for calculating the flexural stiffness of the external restrained units, and further validates the stiffness theory presented in this paper through experiments.

Funding

This research was funded by the Science and Technology Development Plan Project of Jilin Province, China, grant number: YDZJ202401594ZYTS.

The definitions of all parameter symbols adopted in this study are presented in Nomenclature.

Nomenclature

h_{ud}	the width of the upper (and lower) restrained cover plates
t_{ud}	the thickness of the upper (and lower) restrained cover plates
h_{cp}	the height of the left (and right) lateral restrained units
t_{cp}	the thickness of the web plate of the left (and right) lateral restrained units
b_{cp}	the flange width of the left (and right) restrained units
t_{cpb}	the flange thickness of the left (and right) restrained units
h_{cp1}	the distance from the bolt center to the neutral axis of the flange
b_{ud1}	the distance from the bolt center to the mid-plane of the upper (and lower) restrained cover plates
h_x	the distance between the neutral axes of the left and right restrained units
EI_{cpy}	the bending stiffness of the left (and right) restrained units
A_{cp}	the cross-sectional area of the left (and right) restrained units
EI_{udy}	the bending stiffness of the upper (and lower) restrained cover plates
A_{ud}	the cross-sectional area of the upper (and lower) restrained cover plates
l_{bd}	the spacing between web members corresponds to the distance between adjacent bolts
EI_{by}	the bending stiffness of the web members
q_x	the sinusoidal lateral compressive force
L	the length of the unit?
v_0	the initial imperfection value
N	the axial force of the inner core plate
N_{crx}	the overall buckling load of the inner core plate
g_x	the unilateral gap value along the x-direction between the inner core plate and the peripheral restrained units

C_{qx}	the maximum mid-span bending moment obtained from the second-order elastic analysis
V_{0x}	the concentrated lateral force V_{0x} exerted at the end of the units
γ_1	the parameters related to the stiffness reduction coefficient are defined
I_{cp}	the force transfer model of the left (and right) restrained units is simplified as the moment of inertia of the member
I_{ud}	the force transfer model of the upper (and lower) restrained cover plates is simplified as the moment of inertia of the member
γ	the parameters related to the stiffness reduction coefficient are defined
μ_y	the stiffness reduction coefficient of the external Restrained Units
I_y	the moment of inertia of the external Restrained Units as a whole about the Y-axis
h_{cpy}	the effective plate width of the left (and right) restrained units
α_{cp}	the effective width coefficient for the effective plate area on the left (or right) restrained units
Δ_{cp}	the longitudinal deformation induced by the longitudinal force F in the left (or right) restrained units
h_{udy}	the effective plate width of the upper (and lower) restrained cover plates
α_{ud}	the effective width coefficient for the effective plate area on the upper (or lower) restrained cover plates
Δ_{ud}	the longitudinal deformation induced by the longitudinal force F in the upper (or lower) restrained cover plates
Δ	the total in-plane deformation of the restrained plates $\Delta = \Delta_{ud} + \Delta_{cp}$
δ_y	the coupling flexibility coefficient
F	a pair of opposing forces
F_l	the force acting on each bolt group
δ_x	the flexibility coefficient of the bolted connection
l_{bd}	the longitudinal bolt spacing
α_{cp1}	the coefficient for the effect of longitudinal bolt spacing on the left (and right) restrained units
α_{cp2}	the coefficient for the effect of bolt section position on the left (and right) restrained units
α_{cp1}^l	the influence factor of the left (and right) restrained units considering the effect of longitudinal bolt spacing l_{bd}
α_{cp2}^s	the influence factor of the left (and right) restrained units considering the effect of bolt section position h_{cp1}
α_{ud1}	the coefficient for the effect of longitudinal bolt spacing on the upper (and lower) cover plates
α_{ud2}	the coefficient for the effect of bolt section position on the upper (and lower) cover plates
α_{ud1}^l	the influence factor of the upper (and lower) cover plates considering the effect of longitudinal bolt spacing l_{bd}
α_{ud2}^s	the influence factor of the upper (and lower) cover plates considering the effect of bolt section position h_{ud1}
μ_{0y}	the stiffness reduction coefficient due to rotational deformation
α_{01}	the influence of bolt spacing on the reduction coefficient μ_{0y}
α_{02}	the influence of the bolt's cross-sectional position on μ_{0y}
v	the maximum mid-span deflection

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DAMAGE CONTROL AND OPTIMAL DESIGN OF EMBEDDED CFRP CIRCULAR TUBE CFSST COLUMN-H-BEAM JOINT WITH ENERGY-DISSIPATION ANGLE STEEL

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ABSTRACT

Maintaining the integrity of the main structure during seismic events and enabling rapid restoration of its normal functionality afterwards are key design objectives in modern earthquake engineering. Based on the design concept of controllable damage and replaceable components after an earthquake, this study investigates a novel beam-column joint, consisting of an embedded carbon fiber-reinforced polymer (CFRP) circular tube concrete-filled square steel tube (CFSST) column and an H-beam equipped with energy-dissipation angle steel components (EDASCs). A numerical model was established using ABAQUS finite element software, and its accuracy was validated against typical experimental data. Joint models with varying material strengths of EDASCs, design bearing capacity coefficients, and free deformation lengths of spliced energy-dissipating beam sections (SEDBSs) were developed. The influence of various parameters on the seismic performance of beam-column joint and the functionality of “structural fuse” was investigated, leading to optimized design recommendations. The results show that during the plastic stage, the EDASCs accounted for 91% of the total energy dissipation of the joints, effectively protecting the main structure and achieving the design objectives of recoverability and replaceability. The material strength and thickness of the EDASCs significantly affect the seismic performance of the joint. In consideration of the bearing capacity, ductility, and energy dissipation ratio of the joint, Q235 steel is recommended as the material for the EDASCs. The design bearing capacity coefficient of the beam-column joint should range from 0.75–0.98 to achieve optimal seismic performance and fulfill the “structural fuse” function. The free deformation length of the SEDBSs should be set at 1–1.5 times the width of the beam flange.

ARTICLE HISTORY

Received: 11 September 2025
Revised: 3 January 2026
Accepted: 4 January 2026

KEYWORDS

CFRP circular tube;
Concrete-filled square steel tube;
Energy-dissipation angle steel component;
Outer diaphragm;
Design bearing capacity coefficient

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1. Introduction

Concrete-filled steel tube (CFST) members are widely used in super high-rise buildings and large-span spatial structures due to their excellent mechanical properties and economic benefits [1]. To address the issue of the relatively weak lateral confinement of concrete by square steel tubes in CFST members, Li G C [2–4] proposed a composite member of square steel tube with an embedded CFRP circular tube and systematically studied its mechanical properties under different working conditions. The research results show that the confinement effect of the CFRP circular tube on the core concrete can significantly enhance the bearing capacity of the square steel tube concrete composite member and improve its ductility. Currently, the research on square steel tube concrete members with embedded CFRP circular tubes is relatively mature, but the corresponding research on beam-column joints is still insufficient. To promote the application of this structural system, it is urgent to develop high-performance beam-column joints that match it.

As a critical load transfer component in CFST structures, the ductility, energy dissipation capacity, and post-earthquake reparability of beam-column joint are key factors determining the collapse resistance and lifecycle economy of the entire structure under rare earthquakes. In 2011, the seismic damage investigation following the M6.3 earthquake in Christchurch, New Zealand, revealed that numerous bolted-welded hybrid joints experienced brittle fractures in the flange weld areas due to the lack of a graded energy dissipation mechanism [5,6]. This phenomenon promoted the transformation in the seismic design concept of building structures from “collapse prevention design” to “recoverable design” [7–10], leading to the development of a two-stage elastic seismic design method based on performance. In the first stage, replaceable energy dissipation elements are employed to dissipate seismic energy, ensuring that the main structure remains in an elastic state; in the second stage, rapid structural function recovery is achieved by promptly replacing damaged components [11].

To enable rapid recovery of normal functionality for building structures after earthquakes, the development of recoverable and replaceable beam-column joints can be categorized into three stages. In the first stage, the application of various dampers to steel beam-column joints has demonstrated significant enhancement in both energy dissipation capacity and damage control ability within structural systems [12–14], laying a foundation for post-earthquake functional recovery. In the second stage, replaceable energy-dissipation components such as flange cover plates, web connecting plates, and

angle steel components are employed, along with refined structural parameter controls [15–19]. This approach ensures that plastic damage is concentrated on the energy dissipation components, allowing post-earthquake repairs to focus solely on replacing these components. Studies indicate that replacing energy dissipation elements after quasi-static tests typically requires approximately 2 hours [20–24]. In the third stage, a fully bolted prefabricated connection system has been developed, utilizing a graded yield design strategy to ensure that the energy dissipation plate yields first. Both experimental results and finite element analyses confirm that the plastic damage of the joints is effectively concentrated on the energy dissipation plate while the main structure remains in an elastic state [25–30].

Based on the above research, post-earthquake recoverable and replaceable beam-column joints primarily dissipate seismic energy through dampers, end plates, cover plates, connecting plates, and angle steels as “structural fuses” [9]. However, several key challenges remain: First, as the primary energy dissipation component, damper structures are complex, and dampers located outside the beam flange conflict with the enclosure systems, severely limiting the advantages of standardized construction in prefabricated structures. Second, replaceable energy dissipation elements are prone to fatigue failure under rare earthquakes, and their premature failure can aggravate the second-order effect ($P - \Delta$ effect) of gravity on the structural system. Therefore, an in-depth study of the cooperative working mechanisms between the structural parameters and material properties of energy dissipation elements in beam-column joints is necessary.

In order to address the aforementioned challenges, this study proposes a beam-column joint with a replaceable SEDBSs by integrating embedded CFRP circular tube concrete-filled square steel tube (CFSST) column [31], as shown in Fig. 1. This joint incorporates EDASCs as a centralized energy-dissipating component, enabling rapid post-earthquake functional recovery through the replacement of only these elements, thereby substantially reducing repair time and costs. In this study, the general-purpose finite element software ABAQUS is employed to develop a numerical model of the joint, whose accuracy is validated against representative experimental data. Parametric analyses are conducted to evaluate the influence of key parameters on the seismic performance of the joint and on the effectiveness of the “structural fuse” mechanism, leading to practical recommendations for structural optimization that can serve as a reference for the engineering application of resilient and recoverable structural systems.

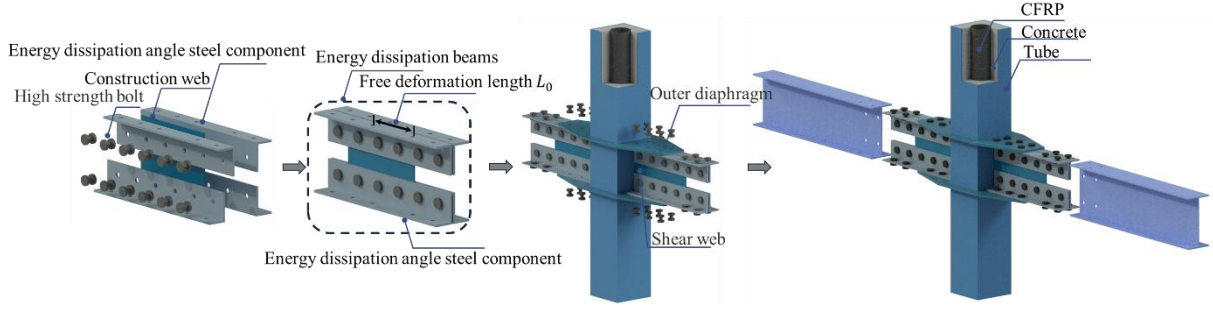


Fig. 1 Schematic diagram of the joint installation process

2. FE modeling and experimental verification

2.1. Joints design

The joint structure consists of an embedded CFRP circular tube CFSST with outer diaphragms and shear webs, a SEDBSs composed of EDASCs and constructed webs, and an H-beam, as shown in Fig. 1. All components are prefabricated in the factory. At the construction site, the assembly of the SEDBSs is completed first. Then, they are connected to the outer diaphragm and shear webs through full bolt connections to form a temporary support system. Finally, the overall hoisting of the H-beams is carried out. The installation is convenient and conducive to rapid disassembly and replacement after an earthquake. The research focuses on the side columns of the standard frame layer, and the beam-column dimensions are extracted from the inflection point of the prototype structure. The prototype structure is a 28-story, 106 m high aluminum alloy inner-core buckling-restrained brace (BRB)-embedded CFRP circular tube CFSST column structural system. The dimensions of the components in the model are as follows: the square steel tube column has a height of 1550 mm and a cross-sectional dimension of 250 mm × 250 mm × 8 mm; the H-beam has a cross-section of 300 mm × 150 mm × 6.5 mm × 9 mm. The inner diameter of the CFRP tube is 160 mm, consisting of three layers. A high-strength friction-type M20 bolt of grade 10.9 is used, with a preload force of 155 kN and a bolt hole diameter of 22 mm, as specified by the design values in the “GB 50017-2017 Standard for Steel Structure Design” [32]. Q355 steel is adopted for both the square steel tube and the H-beam, and the concrete strength grade is C50. The geometric configuration of the model is illustrated in Fig. 2.

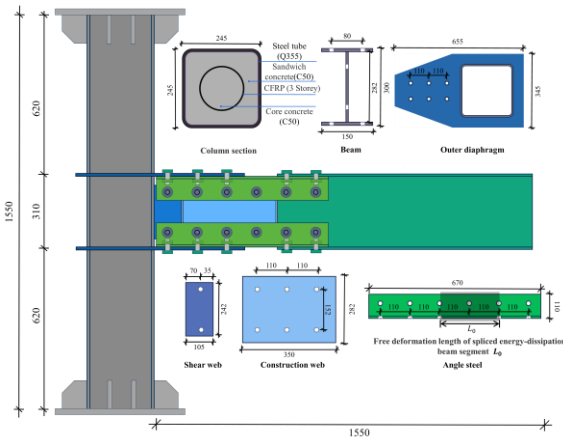


Fig. 2 Details dimensions of numerical models (mm)

2.2. Mechanism analysis

The design objectives of the embedded CFRP circular tube CFSST column-H-beam joint with energy-dissipation angle steel are as follows: (1) Under seismic loading, plastic damage is concentrated on the EDASCs, ensuring that the main structure remains elastic; (2) After an earthquake, the joint can be quickly repaired by replacing the EDASCs, and the seismic performance of the repaired joint is essentially the same as that of the undamaged joint. To achieve design goals and ensure that the EDASCs yields preferentially while dissipating energy intensively, it is necessary to satisfy the strength gradient control

criterion: the plastic resistance bending moment of the EDASCs at the free deformation length of the SEDBSs must be smaller than that of outer diaphragm and beam. The design bearing capacity coefficient α_c is introduced to quantify this index, as shown in Eq. (1):

$$\alpha_c = \frac{M_{pl,fuse}}{M_{pl,beam}} \quad (1)$$

Where: $M_{pl,fuse}$ is the plastic resistance bending moment of EDASCs, $M_{pl,beam}$ is the plastic resistance bending moment of the whole section of the beam.

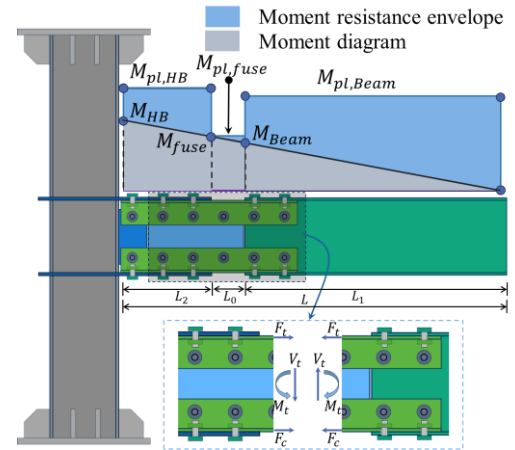


Fig. 3 Mechanism analysis of specimens

In Fig. 3, $M_{pl,HB}$ is the plastic resistance bending moment of the entire section at the connection between the outer diaphragm and the column flange, M_{HB} is the elastic resistance bending moment at the connection between the outer diaphragm and the column flange, M_{fuse} is the elastic resistance bending moment of the EDASCs at the free deformation length of the SEDBSs, and M_{Beam} is the elastic resistance bending moment of the beam. The plastic resistance bending moment of the EDASCs is equivalent to a pair of couple formed by the tension and compression in the upper and lower EDASCs, and the force analysis is shown in Fig. 3. $M_{pl,fuse}$ and $M_{pl,beam}$ can be calculated by Eqs. (2) and (3).

$$M_{pl,fuse} = W_{fuse} f_{y,a} \quad (2)$$

Where: $f_{y,a}$ is the yield strength of EDASCs, W_{fuse} is the elastic section modulus of free deformation length of the SEDBSs.

$$M_{pl,Beam} = W_{Beam} f_{y,Beam} \quad (3)$$

Where: W_{Beam} is the elastic section modulus of beam, $f_{y,Beam}$ is the yield strength of beam.

To ensure the “structural fuse” function of the joint, the following strength criteria must be satisfied: (1) The plastic resistance bending moment at the connection between the outer diaphragm and the column flange, as well as the plastic resistance bending moment of the entire interface of the beam, should be

greater than their respective elastic resistance bending moments; (2) The plastic resistance bending moment of the EDASCs should be less than its elastic resistance bending moment. The above parameters can be calculated by Eqs. (4)–(6):

$$M_{pl,HB} > M_{HB} = P(L_1 + L_0 + L_2) \quad (4)$$

$$M_{pl,Beam} > M_{Beam} = PL_1 \quad (5)$$

$$M_{pl,fuse} < M_{fuse} = P(L_1 + L_0) \quad (6)$$

Where: L_0 is the free deformation length of the spliced link, L_1 is the length of the beam, L_2 is the length from the end of the outer diaphragm to the column flange.

2.3. Unit selection and constitutive relation

The mesh division and boundary conditions of the model are shown in Fig. 4. The CFRP circular tube is modeled using the reduced-integration 4-node shell element (S4R). In the thickness direction of the shell, three Simpson integration points are adopted. The other components adopt the 8-node hexahedron linear reduced-integration element (C3D8R). The grid size of the beam-column members in both the joint area and the non-joint area is 5 mm; To simulate the lateral restraint provided by the floor slabs and secondary beams to the main beams in the frame and prevent the steel beams from experiencing out-of-plane instability, lateral constraints (Limit its displacement in X and Y directions and rotation in X and Z directions) are applied to the steel beams, while Z-direction displacement is released at the top of the column to apply an axial force with an axial compression ratio of 0.3.

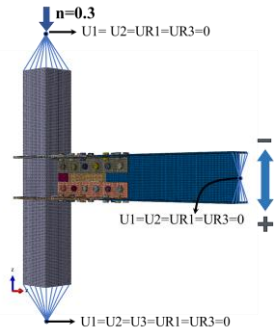


Fig. 4 Boundary conditions and meshing

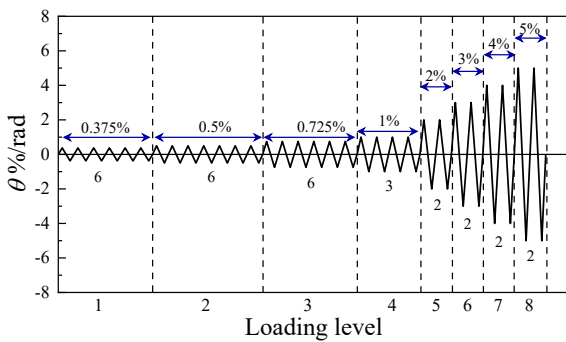


Fig. 5 Loading system

The constitutive relation of concrete adopts a plastic damage model considering the constraint effect of the steel tube, while that of steel adopts a five-segment linear model considering the strengthening section [33]. The CDP model parameters are shown in Table 1. The number of layers of CFRP circular tubes and direction of fiber stress are defined by the composite layer. The elastic-stage stress behavior is simulated using the Lamina model, while brittle fracture behavior at ultimate load is simulated using the Hashin damage function. The basic parameters of CFRP materials are shown in Table 2. The constitutive relation curve of high-strength bolts adopts a multi-linear isotropic hardening model, and bolt strength is calculated according to the standard GB/T 3098.1-2010 [34].

Table 1
CDP model parameters

Dilation Angle	Eccentricity	f_{bo}/f_{co}	K	Viscosity Parameter
40	0.1	1.2	0.6667	0.0005

Table 2
Basic parameters of CFRP materials

Thickness of single-layer CFRP (mm)	Ultimate tensile strength (MPa)	ν	E (GPa)
0.167	4200	0.304	283

2.4. Contact relationship and loading system

The failure mode of the joint is characterized by the formation of a plastic hinge in the SEDBS. The beam-end loading method is beneficial for studying the seismic performance of the side-column joint model. Reciprocal displacement loading is applied at the beam end, following the provisions outlined in the Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-10) [35]. The maximum interlayer displacement angle is set to 5%, satisfying the AISC 341-10 requirement that the maximum plastic rotation angle for high-ductility joints be no less than 0.04 rad. The loading system is presented in Fig. 5.

Under low-cycle reciprocating loading at the beam end, friction occurs between the bolt head and the steel plate, while the bolt shank and the hole wall undergo continuous bearing and separation under cyclic loading. Therefore, the normal contact interaction between the bolt and the steel plate is defined as “hard” contact, with a tangential friction coefficient of 0.45. The tangential interaction between the concrete and the steel tube wall is assigned a friction coefficient of 0.60. Tie constraints are applied at the welded joints to simulate full continuity. Based on the coordinated deformation behavior of CFRP and concrete [36], the two materials are connected using tie constraints to ensure displacement compatibility.

2.5. FE verification of embedded CFRP circular tube CFSST column

This research employs the proposed modeling method to analyze specimen 1200-5-2 from the referenced literature [36], with the computational results juxtaposed against the experimental findings, as illustrated in Fig. 6. The finite element analytical curve aligns well with the test curve, exhibiting a peak bearing capacity discrepancy of 1.77%. Both the experimental specimens and those from finite element simulations exhibited drum-shaped failure. This work presents a modeling method that exhibits remarkable accuracy in simulating CFSST columns with embedded CFRP circular tubes.

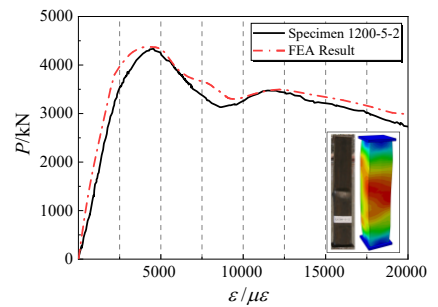


Fig. 6 1200-5-2 load-strain curves comparison

2.6. FE verification of joint

The specimen JD-2 in reference [37] was modeled using the method proposed in this paper. The comparison between the calculation results and the test results is shown in Fig. 7(a). During the test, the shear force generated at the connection between the outer diaphragms and the beam flange exceeded the preload of the high-strength bolts, causing bolt slippage and resulting in a distinct pinching effect in the hysteretic curve. Since the finite-element model did not explicitly simulate the slippage behavior between the bolts and the hole walls, the joint exhibited a higher overall stiffness at the initial stage of loading, leading to a weaker pinching effect and a relatively higher yield load in the calculated hysteretic curve. Despite this, the failure mode was highly consistent between the test and the finite-element modeling: local buckling occurred at the

weak section of the beam flange, and significant plastic deformation was observed around the bolt holes. Additionally, although there was a 19.8% difference in the yield load, the difference in peak bearing capacity was only 3.8%, indicating that the modeling method has high accuracy in simulating the peak bearing capacity performance of this joint.

The specimen RT-2 from reference [38] was modeled using the numerical modeling approach proposed in this paper. A comparison between the calculated and experimental results is presented in Fig. 7(b). In terms of failure mode, both the test and the finite element simulation exhibit consistent behavior: the cantilever and mid-span segments of the steel beam remain elastic, while plastic deformation is localized in the web of the T-shaped steel section. The yield load

differs by 12.4%, and the peak bearing capacity differs by 4.7%, indicating that the discrepancies at key characteristic points are relatively small.

The specimen PRBS in reference [39] was modeled using the method proposed in this paper. The comparison between the calculation results and the test results is shown in Fig. 7(c). From the failure mode, both the test and the finite element simulation show the same failure mode: the dog-bone-shaped beam section near the cantilever beam side flange buckles. The difference in the yield load is 6.2%, and the difference in the peak bearing capacity is 9.3%, indicating that the error of the characteristic points is relatively small. In summary, the proposed simulation method has good accuracy in simulating such nodes.

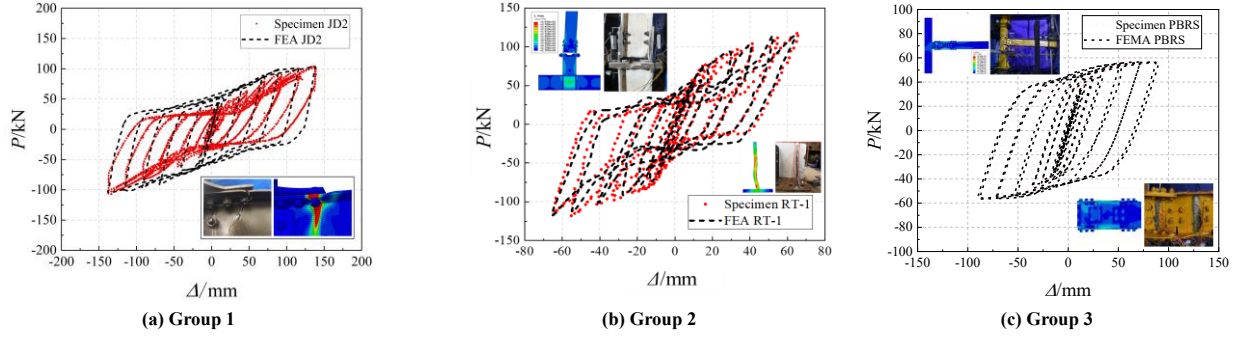


Fig. 7 Joints hysteresis curve comparison

3. FE analysis of material with different EDASCs

This study builds three comparison models (JD0 series) using LY160, Q235, and Q355 steels to elucidate the impact of the material strength of EDASCs on the damage control mechanism and seismic performance of joints. These models are developed under the premise of an equal design bearing capacity coefficient ($\alpha_c = 0.83$) and a fixed free deformation length of the SEDBS ($L_0 = 1.5b_f$). The geometric parameters of the EDASCs for each model are determined based on an equivalent criterion of material strength and thickness, ensuring that EDASCs with different material strengths possess the same bending resistance capacity. Detailed parameters for each model are presented in Table 3.

Table 3
JD0 series specimen parameters

Specimen	material strength	L_0	α_c	b_{JG}/mm
JD0-AS1	LY160	$1.5b_f$	0.83	13
JD0-AS2	Q235	$1.5b_f$	0.83	8
JD0-AS3	Q355	$1.5b_f$	0.83	5

Note: α_c is design resistance capacity coefficient; b_{JG} is EDASCs thickness; b_f is beam flange width; L_0 is free deformation length.

Fig. 8 shows the hysteresis curve of the JD0 series model. The abscissa is the interlayer displacement angle θ , and the ordinate is the actual bearing capacity coefficient α_r of the beam end. The expression is as follows:

$$\alpha_r = \frac{M_r}{M_{pl,beam}} \quad (7)$$

Where: M_r is the actual bending moment of the beam end, which equals to the product of the beam end load (P) and the beam length (L), $M_{pl,beam}$ is the plastic resistance bending moment of the entire beam section.

As illustrated in Fig. 8, the peak values of the actual bearing capacity

coefficients for the JD0-AS1 and JD0-AS2 models are 1.59 and 1.58, respectively, significantly higher than the design bearing capacity coefficients of the joints ($\alpha_c = 0.83$). However, the actual bearing capacity coefficient of the JD0-AS3 model is 1.34, although slightly higher than the design bearing capacity coefficient, the bearing capacity significantly degrades once the interlayer displacement angle exceeds 3%.

The characteristic parameters of the skeleton curve for JD0 series models are consolidated in Table 4. With the same design bearing capacity coefficient, the thickness of the JD0-AS3 model is reduced due to the enhanced material strength of the EDASCs. The initial rotational stiffness (K_0) of the JD0-AS3 model is 19.15% and 10.74% inferior to that of the JD0-AS1 and JD0-AS2 models, respectively, while the ductility (μ) is diminished by 36.78% and 30.71%. It can be deduced that maintaining a consistent design bearing capacity coefficient while enhancing the material strength of the EDASCs substantially diminishes the joint's initial rotational stiffness, bearing capacity, and deformation capacity.

Fig. 9 shows the calculation method for the local hysteresis curve of the EDASCs. The abscissa Δ is the tensile and compressive displacement at point A on the upper flange of the EDASCs, while the ordinate F is the equivalent axial force of the $a - a'$ section of the EDASCs. To reveal the working mechanism of the EDASCs as a “structural fuse”, the local hysteresis curve is extracted, as shown in Fig. 10. Under reciprocating loads applied at the beam end, the EDASCs experiences positive tension elongation and negative compression buckling. In the elastic stage, the positive and negative displacements of the hysteresis curve are symmetrical, and the relationship between the equivalent axial force and displacement is linear, indicating that the EDASCs remains in a linear elastic state without cumulative damage. In the plastic stage, however, the effective bearing area decreases due to buckling deformation under compression, leading to significant pinching of the hysteresis curve.

In summary, the synergistic effect of material strength and thickness of EDASCs is critical for regulating the function of the “structural fuse”. When the ratio of the actual bearing capacity coefficient of the joint to the design bearing capacity coefficient approaches 1, the safety margin of the joint decreases, and the buckling deformation of the flange of the EDASCs increases, resulting in a deterioration of the joint's bearing capacity and a significant reduction in its deformation capacity. The “structural fuse” function fails earlier than expected.

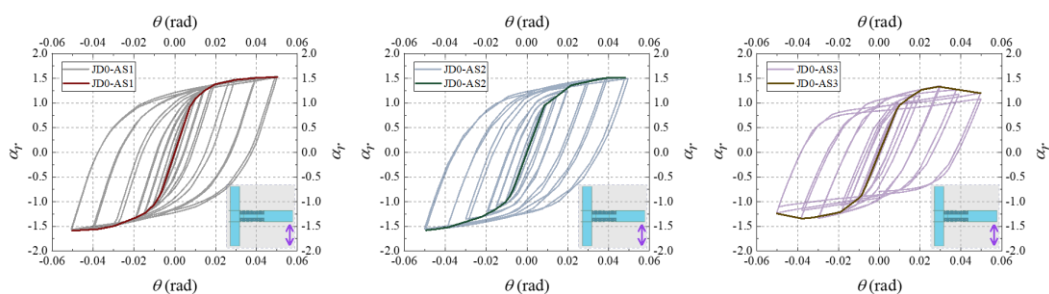


Fig. 8 Comparison of the global hysteresis curves of joints

Table 4
Summary of characteristic parameters of skeleton curves

Specimen	K_0 (kN•m/rad)	α_r	α_c	α_r/α_c	μ
JD0-AS1	19812.01	1.59	0.83	1.92	5.71
JD0-AS2	17945.39	1.58	0.83	1.90	5.21
JD0-AS3	16018.57	1.34	0.83	1.61	3.61

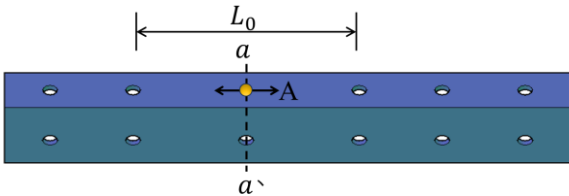


Fig. 9 Calculation method of horizontal and vertical coordinates for local hysteresis curves

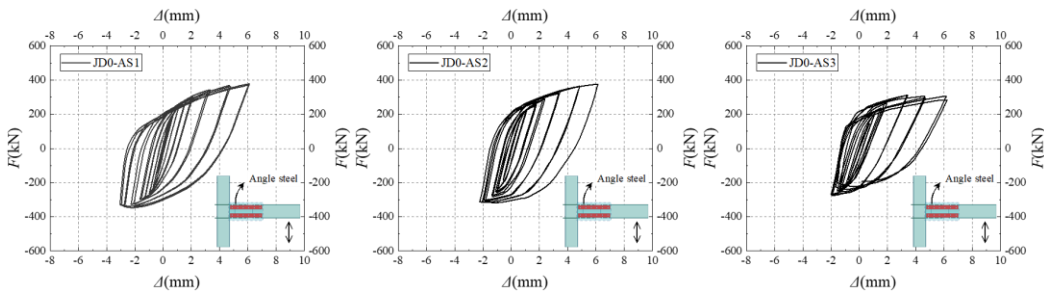


Fig. 10 Comparison of the local hysteresis curves of EDASCs

Fig. 11 shows the ratio of the total energy dissipation of each component to that of the joint model. In the post-processing module of the ABAQUS software, extract the ALLPD (plastic energy dissipation) values of each component. In Fig. 11, E_D , E_{JG} , E_{HB} , E_H , E_{FB} , E_{JQB} , and E_C correspond to the energy dissipation values of the entire joint, EDASCs, outer diaphragm, beam, constructed webs, shear web and column respectively. As shown in Fig. 11, during the elastic stage, the total energy dissipation of the column (E_C) and the EDASCs (E_{JG}) is close to 100%. In the plastic stage, the EDASCs becomes the primary energy dissipation component. When the interlayer displacement angle

reaches 5%, the energy dissipation ratio of the JD0-AS2 model's EDASCs increases to 98%, while that of the column decreases to 1.7%. Compared with models JD0-AS1 and JD0-AS3, the energy dissipation of the JD0-AS2 model's EDASCs increases by 11.34% and 8.16%, respectively. Fig. 12 shows a comparison of each component's cumulative energy dissipation capacity. JD0-AS2 model's cumulative energy dissipation value is 87.85 kN•m, which is higher than those of JD0-AS1 ($E_D = 81.24$ kN•m) and JD0-AS3 ($E_D = 80.39$ kN•m) by 8.13% and 9.28%, respectively.

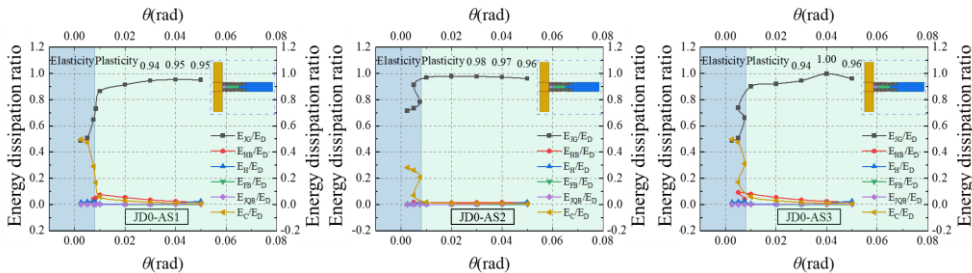


Fig. 11 Energy dissipation ratio

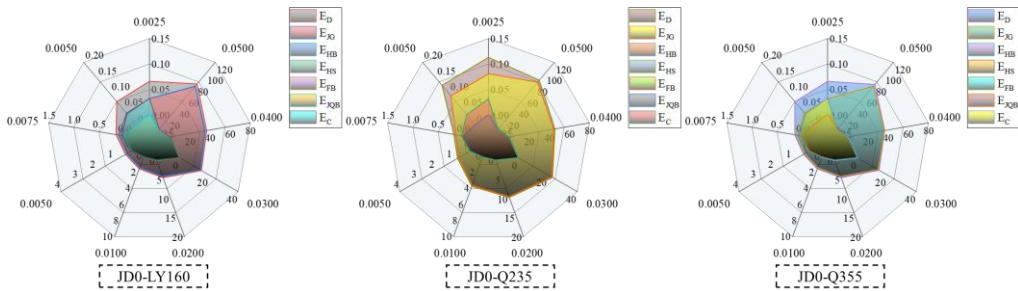


Fig. 12 Cumulative energy dissipation

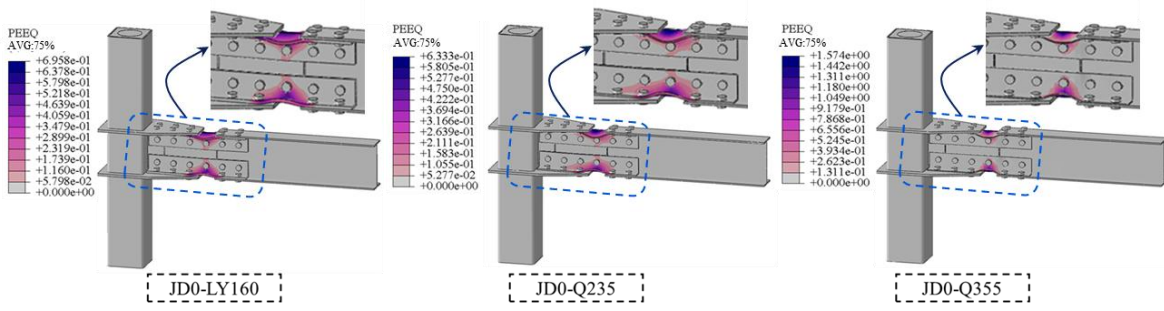


Fig. 13 Damage distributions

Fig. 13 shows the equivalent plastic strain (PEEQ) distribution of each joint model. Under the same design bearing capacity coefficient ($\alpha_c = 0.83$), the plastic damage in the joint is primarily concentrated on the EDASCs, while the remaining components are in elastic state. In summary, the synergistic effect of material strength and thickness of EDASCs significantly influences the seismic performance of joints. Although the LY160 steel has features such as a low yield point and high ductility, which are conducive to enhancing the energy dissipation capacity of the joint, its relatively low yield strength leads to a relatively lower ultimate bearing capacity of the joint. However, the joints using EDASCs made of Q235 steel not only have good ductility and stable energy dissipation capacity, but also exhibit higher ultimate bearing capacity and stiffness reserve, which is more in line with the optimization design principle of this paper aiming at “structural fuse” function and seismic performance. Therefore, Q235 steel is a better choice that takes both performance and cost into account.

4. Parameter analysis

According to the study results in Section 3, Q235 steel has been chosen for the EDASCs. In this section, the design bearing capacity coefficient and the free

deformation length of the SEDBSs are identified as the primary parameters. These parameters are used to examine their effects on the “structural fuse” function and seismic performance of the embedded CFRP circular tube CFSST column-H-beam joint with energy-dissipation angle steel.

4.1. Parameter design

The design bearing capacity coefficient α_c is used to characterize the weakening degree of EDASCs, with α_c values set at 0.58, 0.66, 0.75, 0.83, 0.98, and 1.19, respectively. The free deformation length of the SEDBSs is set to be 1, 1.5, and 2 times the width of the beam flange. The length of EDASCs are 610 mm, 670 mm, and 750 mm respectively, corresponding to free deformation lengths of 160 mm, 220 mm, and 300 mm as shown in Fig. 14. The specific parameters of each model are shown in Table 5. The “JD-AS” double parameter number is used: the JD series characterizes the free deformation length of different SEDBSs, while the AS series characterizes different design bearing capacity coefficients. A total of 18 groups of models were established. The loading system, material properties and boundary conditions are consistent with those described in Section 3.

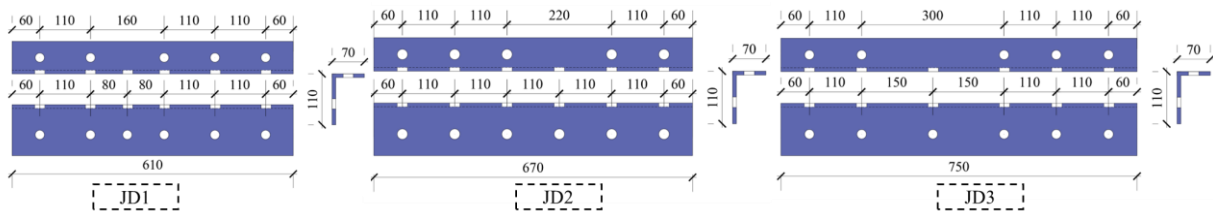


Fig. 14 Detail dimensions of EDASCs (mm)

Table 5
Numbers and main parameters of joint models

Specimen	L_0/b_f	b_{JC}/mm	α_c
JD1-AS1	1	5	0.58
JD1-AS2		6	0.66
JD1-AS3		7	0.75
JD1-AS4		8	0.83
JD1-AS5		10	0.98
JD1-AS6		13	1.19
JD2-AS1	1.5	5	0.58
JD2-AS2		6	0.66
JD2-AS3		7	0.75
JD2-AS4		8	0.83
JD2-AS5		10	0.98
JD2-AS6		13	1.19
JD3-AS1	2	5	0.58
JD3-AS2		6	0.66
JD3-AS3		7	0.75
JD3-AS4		8	0.83
JD3-AS5		10	0.98
JD3-AS6		13	1.19

4.2. Overall hysteresis responses

Fig. 15 shows the hysteresis curves of the joint models. As the design bearing capacity coefficient increases, the shape of the hysteresis loops gradually transitions from “rectangular” to “shuttle-like,” indicating improved stability and reduced degradation of the joint's load-carrying capacity under cyclic loading. Table 6 summarizes the characteristic parameters of the skeleton curves for each model. The actual bearing capacity coefficient exceeds its design value, and the ratio of actual to design bearing capacity coefficient first increases and then decreases with increasing design coefficient. The maximum ratio is observed in the JD-AS4 model among the JD series.

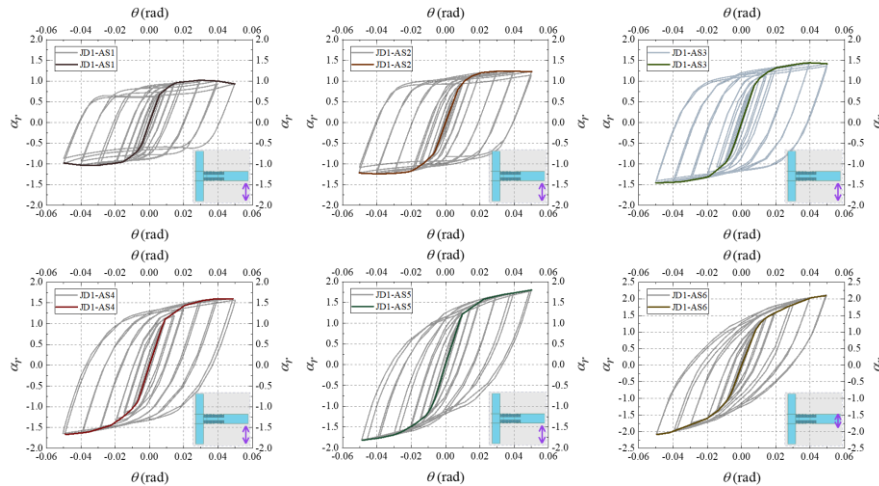
Since the hysteresis curve of the joint does not exhibit obvious stiffness degradation, the shape change specific energy theory is applied to compare the ductility of each specimen. According to this theory, material damage occurs when the shape change ratio at any location within the specimen reaches its limit value under unidirectional stress. As shown in Table 6, the displacement ductility coefficient of the joint ranges from 3.16 to 6.36, indicating a significant capacity for deformation. When the free deformation length of the SEDBSs at the joint remains constant, an increase in the design bearing capacity coefficient

leads to greater EDASC thickness, reduced joint deformation capacity, and higher initial rotational stiffness. Conversely, with a constant design bearing capacity coefficient, increasing the free deformation length of the SEDBSs results in lower initial rotational stiffness and a non-monotonic variation in ductility first increasing and then decreasing. Clearly, both the design bearing capacity coefficient and the free deformation length of the SEDBSs have significant effects on the joint's ductility and initial rotational stiffness. To prevent out-of-plane instability due to excessive weakening, a lower bound should be established for the design bearing capacity coefficient. Moreover, an upper bound for the free deformation length of the SEDBSs is necessary to achieve an optimal balance between the “structural fuse” function and the seismic performance of the joint.

According to American standard ANSI/AISC 360-22 [40] and European standard EC3 [41], joints are classified according to the initial rotational stiffness. For connection joints without support structure, when the initial rotational stiffness K_0 of the joints satisfies $(0.5-25) EI/L$, it is proven to be classified as semi-rigid joints. The calculated initial rotational stiffness of all joint models falls within the range of $(1.42-2.27) EI/L$, confirming their classification as semi-rigid joints.

Table 6
Skeleton curve characteristic parameter summary

Specimen	K_0 (kN•m/rad)	$\frac{K_0}{EI/L}$	Category	α_r	α_c	α_r/α_c	μ
JD1-AS1	17742.80	1.82	Semi-rigid	1.03	0.58	1.78	6.36
JD1-AS2	18606.40	1.90		1.24	0.66	1.88	6.27
JD1-AS3	19525.47	2.00		1.46	0.75	1.95	5.77
JD1-AS4	20011.35	2.05		1.67	0.83	2.01	5.46
JD1-AS5	21289.74	2.18		1.82	0.98	1.86	5.11
JD1-AS6	22191.64	2.27		2.09	1.19	1.76	3.41
JD2-AS1	15758.36	1.61		0.97	0.58	1.67	6.47
JD2-AS2	16934.95	1.73		1.20	0.66	1.82	6.35
JD2-AS3	17444.27	1.79		1.37	0.75	1.83	5.79
JD2-AS4	17945.39	1.84		1.58	0.83	1.90	5.47
JD2-AS5	19273.30	1.97		1.73	0.98	1.77	5.20
JD2-AS6	20138.60	2.06		2.01	1.19	1.69	3.57
JD3-AS1	13889.75	1.42		0.89	0.58	1.53	6.23
JD3-AS2	14800.05	1.51		1.09	0.66	1.65	5.39
JD3-AS3	16450.54	1.68		1.47	0.75	1.96	5.23
JD3-AS4	16604.23	1.70		1.31	0.83	1.58	5.11
JD3-AS5	17328.07	1.77		1.64	0.98	1.67	4.66
JD3-AS6	17975.37	1.84		1.88	1.19	1.58	3.16



(a) $L_0/b_f = 1$, JD1 series

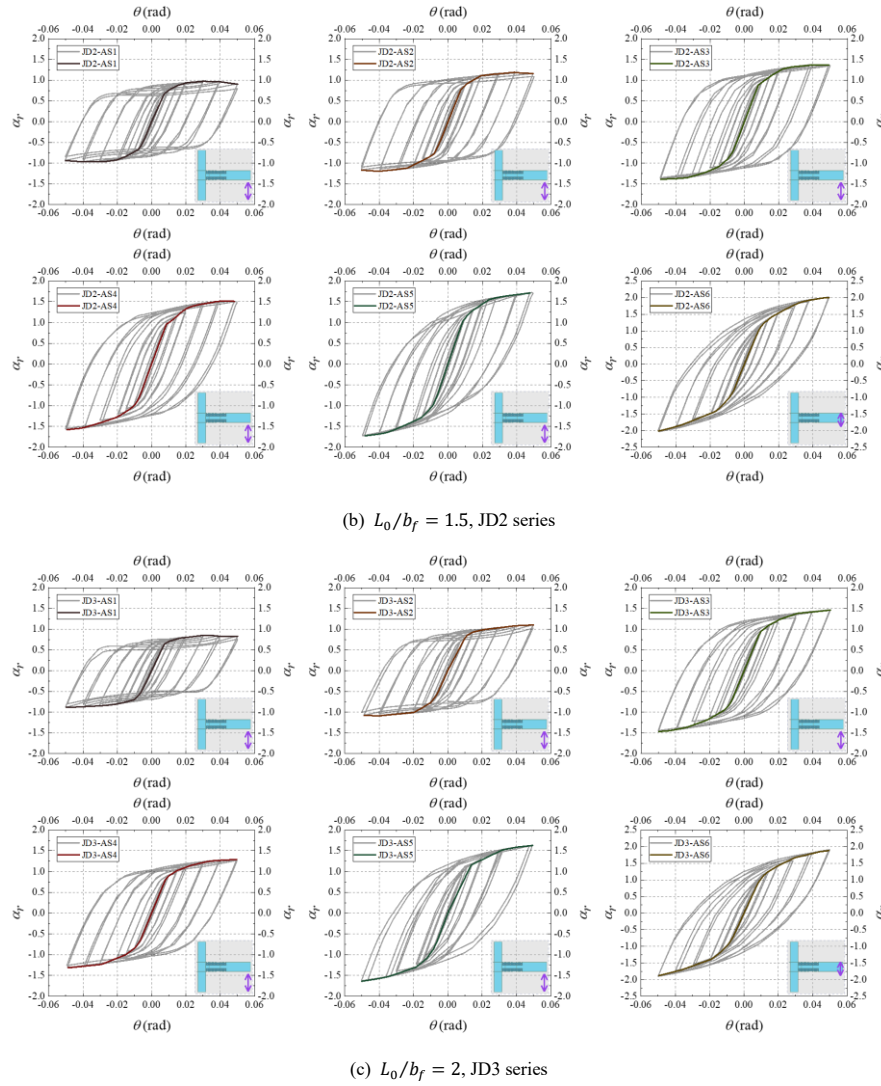


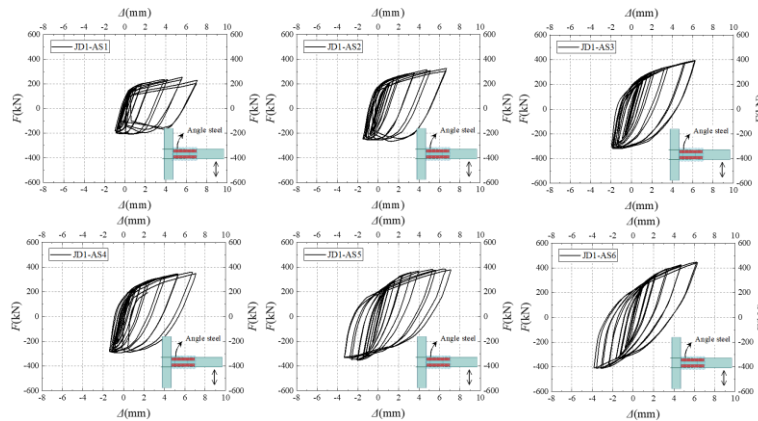
Fig. 15 Comparison of the global hysteresis curves of joints

4.3. EDASCs hysteresis responses

To analyze the working mechanism of EDASCs, the local hysteresis curves of these components in each model are extracted, as shown in Fig. 16. In the elastic stage, positive and negative tension and compression are symmetrically distributed; in the plastic stage, a significant pinching effect occurs. This is because the EDASCs does not yield in the elastic stage, maintaining consistent positive and negative tension and compression deformation. In the plastic stage, the EDASCs is elongated under positive tension, resulting in a full hysteresis curve with good energy dissipation capacity. Under negative compression,

buckling deformation occurs in the EDASCs, leading to reduced compressive deformation displacement and decreased energy dissipation capacity.

Under the condition of a constant free deformation length of the spliced link, as the design bearing capacity coefficient increases, the weakening of the EDASCs decreases, while the compression deformation of the EDASCs increases. This results in an increased proportion of the load transmitted from the bolt to the outer diaphragm, thereby weakening the “structural fuse” function. When the design bearing capacity coefficient is consistent, increasing the free deformation length of the spliced link has minimal influence on the mechanical properties of the EDASCs.

(a) $L_0/b_f = 1$, JD1 series

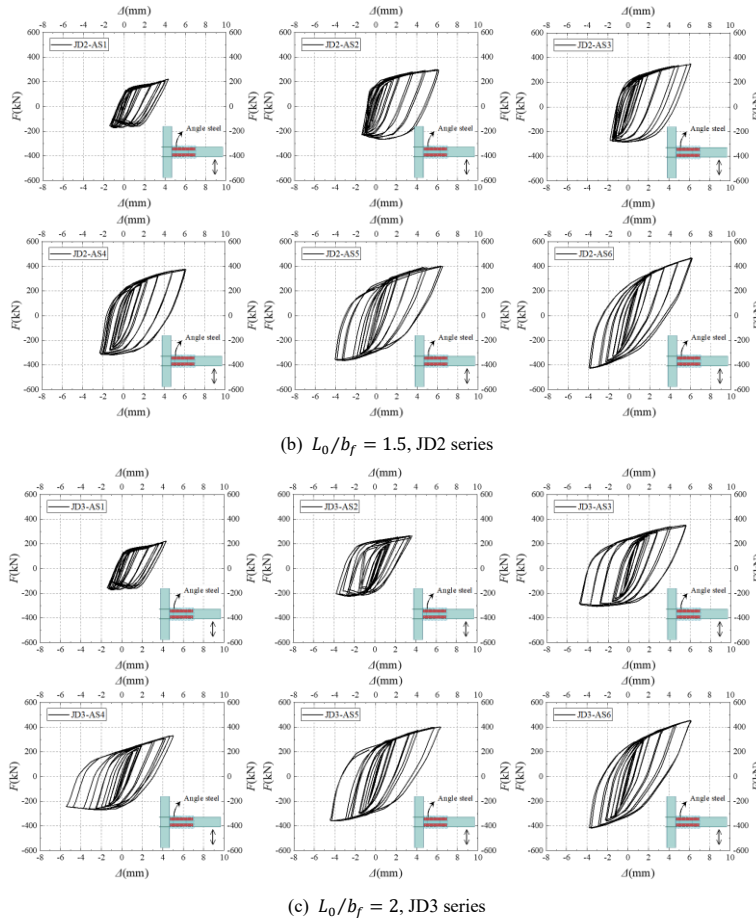


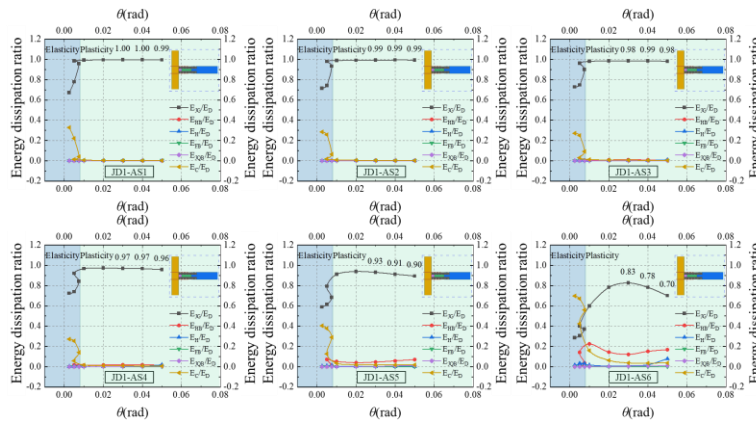
Fig. 16 Comparison of the local hysteresis curves of EDASCs

4.4. Energy dissipation ratio

The ratio of the energy dissipation of each component to the total energy dissipation of the joint under each displacement loading amplitude is extracted to quantitatively evaluate the “structural fuse” function of the EDASCs, as shown in Fig. 17. In the post-processing module of the ABAQUS software, extract the ALLPD (plastic energy dissipation) values of each component. In the elastic stage, energy dissipation is shared between column and the EDASCs; in the plastic stage, energy dissipation is primarily borne by the EDASCs, while the contribution of the column decreases. When the design bearing capacity coefficient is less than 0.75, the energy dissipation ratio of the EDASCs reaches 90%–100% under an interlayer displacement angle of 5%, effectively realizing the “structural fuse” function. When the design bearing capacity coefficient exceeds 0.98, the energy dissipation ratio of the EDASCs decreases to 70%–95%, while that of the outer diaphragms and column increases to 3%–24%. It has been demonstrated that when the design bearing capacity coefficient exceeds 0.98, plastic damage diffuses from the EDASCs to the outer

diaphragms and the beam, causing irreversible plastic deformation in the main structure and weakening the “structural fuse” function of the EDASCs. Increasing the free deformation length of the spliced link increases the energy dissipation ratio of the EDASCs by less than 3%, indicating that the free deformation length of the spliced link has limited influence on the energy dissipation ratio of each component in the model.

In summary, an increase in the design bearing capacity coefficient is associated with a gradual decrease in the energy dissipation ratio of the EDASCs, while the energy dissipation ratio of the column and outer diaphragms gradually increases. This phenomenon observed in the experimental results leads to a weakening of the “structural fuse” function of the EDASCs. Therefore, it is necessary to establish an upper limit for the design bearing capacity coefficient. This will ensure that plastic damage is effectively concentrated on the EDASCs. Furthermore, the augmentation of the free deformation length of the SEDBS exerts a negligible influence on the enhancement of the “structural fuse” function of the EDASCs.



(a) $L_0/b_f = 1$, JD1 series

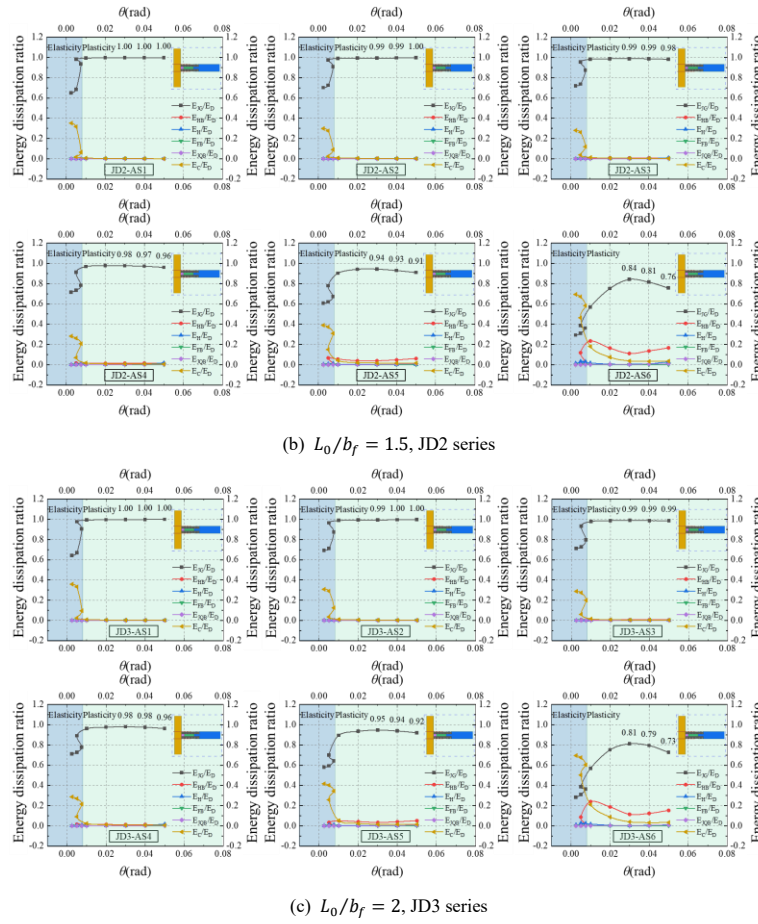
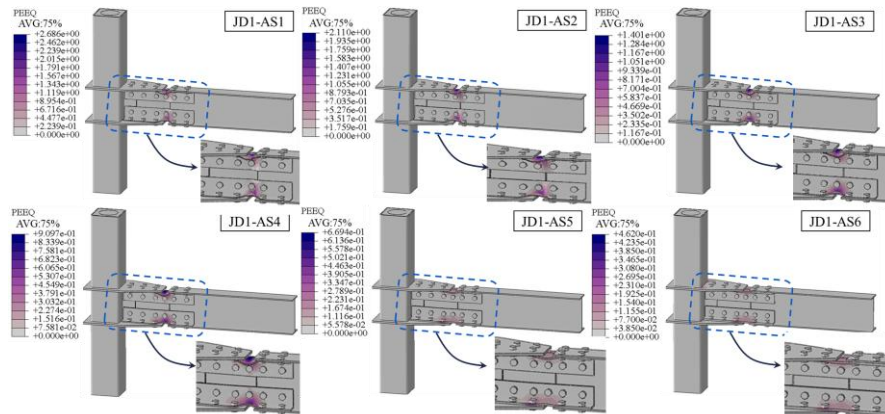


Fig. 17 Comparison of the energy dissipation ratio

4.5. Damage deformation distribution

Fig. 18 illustrates the distribution of equivalent plastic strain (PEEQ) across each model. A darker color indicates a higher level of accumulated plastic damage, reflecting the material's ductility and susceptibility to fracture. Thus, PEEQ serves as an effective index for evaluating the fracture potential of the steel. All EDASCs exhibit significant buckling deformation, with peak equivalent plastic strain localized within the EDASCs. When the design bearing capacity coefficient is below 0.75, reduced thickness of the EDASCs leads to pronounced buckling and diminished out-of-plane stability of the joint.

Conversely, when the coefficient exceeds 0.98, the equivalent plastic strain in the EDASCs decreases, and the plastic hinge shifts away from the beam, outer diaphragms, and column flange connections, partially compromising the intended "structural fuse" behavior. Examination of the columns in Fig. 18 further shows that, under a constant design bearing capacity coefficient, increasing the free deformation length of the spliced link reduces the overall equivalent plastic strain level at the joint and promotes a more uniform distribution of damage within the EDASCs. This indicates that extending the free deformation length of the SEDBSs enhances control over damage localization in the EDASCs.

(a) $L_0/b_f = 1$, JD1 series

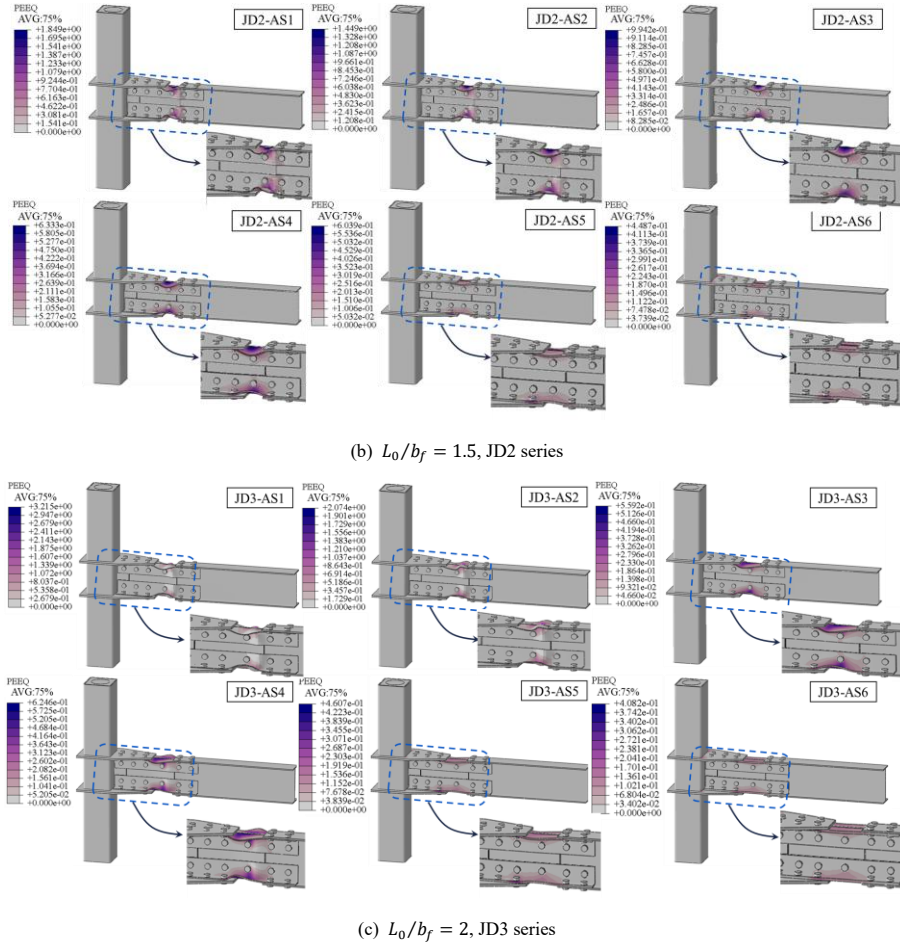


Fig. 18 Comparison of cumulative plastic strain distributions of joints

5. Optimal design suggestion

To attain the design objective of controllable joint damage and post-earthquake reparability, it is crucial to concentrate plastic deformation on the EDASCs. Based on the ratio of the total energy dissipation of the EDASCs, columns, outer diaphragms, and joints under a 5% interlayer displacement angle, combined with the analysis in Section 4, it can be observed that when the energy dissipation ratio of the outer diaphragms and columns exceeds 4% and the energy dissipation ratio of the EDASCs is less than 90%, plastic damage distribution diffuses from the EDASCs to the beam, outer diaphragms, and column flange connections. Consequently, the critical value serves as the effective threshold of the “structural fuse”, ensuring rapid restoration of functionality by replacing the EDASCs post-earthquake.

Fig. 19 illustrates the correlation between the energy dissipation ratio of each component, the design bearing capacity coefficient, and the free deformation length of the spliced link. The analysis of the energy dissipation ratio presented in Table 7 and Fig. 19 reveals the impact of the design bearing capacity coefficient and the free deformation length of the SEDBSs on the energy dissipation mechanism of the joint. At an interlayer displacement angle of 5%, when the design bearing capacity coefficient is below 0.75, the EDASCs

is a thin-walled angle steel component, susceptible to significant out-of-plane buckling deformation, consequently diminishing the joint's load-bearing capacity. When the design bearing capacity coefficient surpasses 0.98, the augmented strength of the EDASCs causes the joint's plastic degradation to propagate from the EDASCs to the beam and outer diaphragms. The energy dissipation ratio of the outer diaphragms rises to 5%-20%, resulting in the failure of the “structural fuse” function. With a consistent design bearing capacity coefficient, augmenting the free deformation length of the SEDBSs leads to variations of up to 3% in the energy dissipation ratios of the outer diaphragms and columns, and up to 5% in the energy dissipation ratio of the EDASCs.

In summary, considering the bearing capacity, initial rotational stiffness, ductility, “structural fuse” function, and material utilization rate of the joints, it is advisable to establish the design bearing capacity coefficient within the range of 0.75-0.98, and the free deformation length of the spliced link beam section should be 1-1.5 times the width of the beam flange. This design guarantees the preferential yielding of the EDASCs. By meticulously adjusting the design bearing capacity coefficient and the free deformation length of the SEDBSs, the design goals of “concentrated energy dissipation in the EDASCs, no damage in the main structure, and rapid replacement after an earthquake” can be achieved.

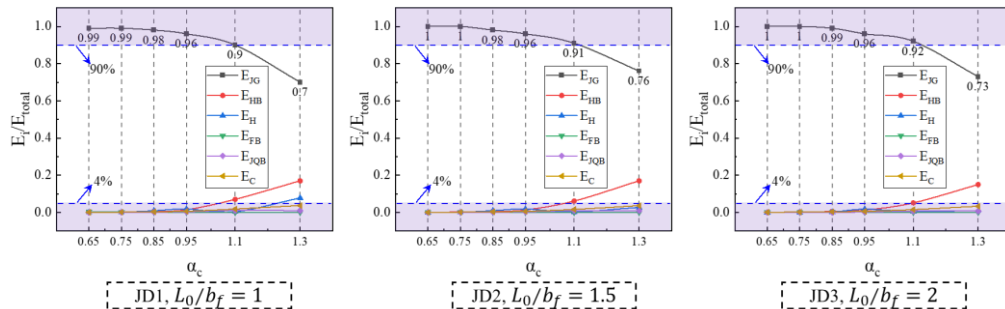


Fig. 19 The relationship between the proportion of component energy dissipation and key parameters

Table 7
Comparison of fuse effect of structure

Specimen	Energy proportion of EDASCs	Energy proportion of column	The proportion of energy dissipation of outer diaphragm	α_r/α_c	Plasticity concentration position
JD1-AS1	99.61%	0.14%	0	1.78	EDASCs
JD1-AS2	99.46%	0.18%	0.29%	1.88	EDASCs
JD1-AS3	98.51%	0.42%	0.81%	1.95	EDASCs
JD1-AS4	96.77%	0.79%	1.82%	2.01	EDASCs
JD1-AS5	91.31%	1.77%	5.76%	1.86	outer diaphragm, EDASCs
JD1-AS6	70.12%	3.65%	16.82%	1.76	outer diaphragm, EDASCs
JD2-AS1	99.82%	0.14%	0	1.67	EDASCs
JD2-AS2	99.51%	0.19%	0.28%	1.82	EDASCs
JD2-AS3	98.63%	0.42%	0.79%	1.83	EDASCs
JD2-AS4	97.18%	0.71%	1.57%	1.90	EDASCs
JD2-AS5	92.91%	1.54%	4.72%	1.77	outer diaphragm, EDASCs
JD2-AS6	75.68%	3.55%	16.59%	1.69	outer diaphragm, EDASCs
JD3-AS1	99.81%	0.15%	0	1.53	EDASCs
JD3-AS2	99.53%	0.22%	0.23%	1.65	EDASCs
JD3-AS3	98.71%	0.36%	0.62%	1.96	EDASCs
JD3-AS4	97.51%	1.28%	0.62%	1.58	EDASCs
JD3-AS5	93.85%	2.09%	3.99%	1.67	outer diaphragm, EDASCs
JD3-AS6	79.28%	3.27%	15.23%	1.58	outer diaphragm, EDASCs

6. Conclusions

This study investigated the seismic performance and “structural fuse” function of the embedded CFRP circular tube CFSST column-H-beam joint with energy-dissipation angle steel under reciprocating load using ABAQUS finite element analysis. The analysis focused on the influence of the material strengths of EDASCs, design bearing capacity coefficients, and free deformation lengths of SEDBSs on energy dissipation, initial rotational stiffness, and damage distribution, and provided optimized design recommendations. The main conclusions are as follows:

(1) Considering bearing capacity, initial rotational stiffness, and energy dissipation ratio of joints, it is suggested that the steel of EDASCs should be Q235, which can effectively balance the “structural fuse” function and seismic performance of beam-column joints. According to European standard EC3, The embedded CFRP circular tube CFSST column-H-beam joint with EDASCs is proven to be classified as a semi-rigid joint.

(2) As the design bearing capacity coefficient increases, the plastic damage distribution diffuses from the EDASCs to the outer diaphragm. The energy dissipation of the column and the outer diaphragm accounts for more than 5%, and the damage cannot be concentrated on the EDASCs. However, when the design bearing capacity coefficient is less than 0.75, the EDASCs is thin-walled, and obvious out-of-plane buckling deformation will occur under large deformation, resulting in serious bearing capacity degradation of the joint. It is suggested that the design bearing capacity coefficient of the joint should be 0.75–0.98.

(3) The increase of the free deformation length of the SEDBSs has a significant effect on the plastic damage distribution of the joint, but the improvement of the “structural fuse” function is limited. When the free deformation length of the SEDBSs exceeds 1.5 times the width of the beam flange, the buckling deformation of the EDASCs decreases, but the bearing capacity, initial rotational stiffness, ductility and energy dissipation capacity of the joint are significantly reduced. It is suggested that the free deformation length of the spliced link should be 1–1.5 times the width of the beam flange.

Acknowledgements

The authors acknowledge the financial support from Key Projects of the National Natural Science Foundation of China, China (No. 51938009), and the Natural Science Foundation of Shenyang, China (No. 23-503-6-09).

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