Advances in Steel Structures (ICASS 2020)

Edited by

Siu-Lai Chan
Department of Civil and Environment Engineering, The Hong Kong Polytechnic University

Zhi-Xiang Yu
School of Civil Engineering, Southwest Jiaotong University

Published by
Hong Kong Institute of Steel Construction Limited
# Table of Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Preface</td>
<td>XI</td>
</tr>
<tr>
<td><strong>Volume I</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Keynote Lectures</strong></td>
<td></td>
</tr>
<tr>
<td>SEISMIC DESIGN AND ANALYSIS OF STEEL PANEL DAMPERS FOR STEEL FRAME</td>
<td>2</td>
</tr>
<tr>
<td>BUILDINGS</td>
<td></td>
</tr>
<tr>
<td><em>Keh-Chyuan Tsai</em> and Chung-Hsiang Hsu</td>
<td></td>
</tr>
<tr>
<td>THE CONTINUOUS STRENGTH METHOD - REVIEW AND OUTLOOK</td>
<td>15</td>
</tr>
<tr>
<td><em>Leroy Gardner</em>, Xiang Yun and Fiona Walport</td>
<td></td>
</tr>
<tr>
<td><strong>Assembled Structure</strong></td>
<td></td>
</tr>
<tr>
<td>A NEW TYPE OF ASSEMBLED THERMAL INSULATION DECORATIVE WALL SYSTEM</td>
<td>28</td>
</tr>
<tr>
<td>FIRE RESISTANCE STUDY</td>
<td></td>
</tr>
<tr>
<td><em>C.L. Wang</em>, S.R. Jiang, B.C. Li and Shuai Li</td>
<td></td>
</tr>
<tr>
<td>RESEARCH ON SEISMIC BEHAVIOR OF ASSEMBLED BEAM-COLUMN JOINTS WITH C</td>
<td>38</td>
</tr>
<tr>
<td>SHAPED CANTILEVER SECTION</td>
<td></td>
</tr>
<tr>
<td>EXPERIMENTAL STUDY AND NUMERICAL ANALYSIS ON SEISMIC BEHAVIOR OF</td>
<td>59</td>
</tr>
<tr>
<td>ASSEMBLED BEAM-COLUMN JOINTS WITH C-SHAPED CANTILEVER SECTION</td>
<td></td>
</tr>
<tr>
<td>RESEARCH ON DYNAMIC LOAD CARRYING CAPACITY OF ASSEMBLED INTERNAL</td>
<td>82</td>
</tr>
<tr>
<td>STIFFENING WIND TURBINE TOWER BASED ON MULTI-SCALE MODELING</td>
<td></td>
</tr>
<tr>
<td><em>F.W. Wang</em>, K.M. Zhou and S.T. Ke</td>
<td></td>
</tr>
<tr>
<td><strong>Bridge</strong></td>
<td></td>
</tr>
<tr>
<td>SOUND RADIATION OF ORTHOTROPIC STEEL DECKS SUBJECTED TO MOVING</td>
<td>93</td>
</tr>
<tr>
<td>VEHICLE LOADS</td>
<td></td>
</tr>
<tr>
<td><em>Y.C. You and X. Zhang</em></td>
<td></td>
</tr>
<tr>
<td>POWER FLOW ANALYSIS OF BRIDGE U-RIB STIFFENED PLATES BASED ON THE</td>
<td>102</td>
</tr>
<tr>
<td>CONCEPT OF STRUCTURAL INTENSITY</td>
<td></td>
</tr>
<tr>
<td><em>D.R. Kong and X. Zhang</em></td>
<td></td>
</tr>
</tbody>
</table>

1
VIBRO-ACOUSTICAL PERFORMANCE OF A STEEL BEAM OF GROOVE PROFILE: FIELD TEST AND NUMERICAL ANALYSIS
Z.Q. Liu and X. Zhang*

PERFORMANCE OPTIMIZATION OF A STEEL-UHPC COMPOSITE ORTHOTROPIC BRIDGE WITH INTELLIGENT ALGORITHM
Z. Xiang*, Z.W. Zhu, J.Y. Cai and J.P. Li

LOAD-CARRYING CAPACITY OF DAMAGED STEEL GIRDER
E. Yamaguchi*, T. Amamoto, D. Nakashima and K. Shiraishi

Cold-Formed

EXPERIMENTAL STUDY ON MECHANICAL PROPERTIES OF STRAW BALE
H.S. Sun, B.Z. Cao*, Z.H. Chen

A SURROGATE MODEL TO ESTIMATE THE AXIAL COMpressive CAPACITY OF COLD-FORMED STEEL OPEN BUILT-UP SECTIONS
S.R. Kho*, A.L.Y. Ng, D.T.W. Looi

LOCAL BUCKLING BEHAVIORS OF COLD-FORMED CIRCULAR HOLLOW SECTIONS HIGH STRENGTH STEEL STUB COLUMNS BASED ON A HIGH-FIDELITY NUMERICAL MODEL
C. Yang, L. Ying* and Y.N. Zhao

BEHAVIOR OF WEB PERFORATED COLD-FORMED STEEL BEAMS UNDER COMBINED BENDING AND SHEAR ACTION
L.P. Wang*, J. Li, X.X. Cao, H.B. Wang

OVERHANG EFFECT ON WEB CRIPLING CAPACITY OF COLD-FORMED AUSTENITIC STAINLESS STEEL SHS MEMBERS: AN EXPERIMENTAL STUDY
K.J. Zhan, C. Chen, Y. Cai and H.T. Li*

Composite

CALCULATION METHOD OF ULTIMATE LOAD BEARING CAPACITY OF CONCRETE FILLED STEEL TUBULAR LATTICE COLUMNS
J.J. Qi*, X. Hu, W.B. Zhou, W.H. Shi and Z. Huang

AXIAL COMPRESSION BEHAVIOR OF SQUARE THIN-WALLED CFST COLUMN TO RC BEAM JOINTS
D. GAN*, Z.X. Zhao, X.H. Zhou and Z. Zhou*
NUMERICAL SIMULATION ANALYSIS OF TEMPERATURE FIELD OF BOX-TYPE COMPOSITE WALL
He Q.Q, Li R, Xue C, Lan T and Qin G.C

251

THERMO-MECHANICAL COUPLING RESPONSE ANALYSIS OF THE BOX-PLATE PREFABRICATED STEEL STRUCTURE UNDER FIRE
Xue C, Li R, Qin G.C and Lan T*

264

STUDY ON FIRE RESISTANCE OF BOX-TYPE COMPOSITE WALLS
Fu Y.Q., He Q.Q., Qin G.C., Lan T.* and Li R.

276

NUMERICAL SIMULATION AND RESEARCH ON WELDING RESIDUAL STRESS OF BOX - TYPE STEEL STRUCTURE
Gao R.X., Men J.J., Lan T* and Li R

288

STUDY ON SHEAR BEHAVIOR OF BOX – TYPE STEEL STRUCTURE CONSIDERING WELDING EFFECT
Wang S, Xue C, Lan T* and Men J.J

296

STUDY ON LOCAL BEARING CAPACITY OF COMPOSITE I-GIRDER WITH CONCRETE - FILLED TUBULAR FLANGE AND CORRUGATED WEB
C.J. Wu, L.X. Deng* and Y.B. Shao

306

PERFORMANCE OF STUD SHEAR CONNECTIONS IN COMPOSITE SLABS WITH VARIOUS CONFIGURATIONS
M.H. Shen, K.F. Chung*, X.D. Wang

312

STUDY OF INITIAL IMPERFECTION OF CONCRETE-FILLED CIRCULAR STEEL TUBE COLUMNS FOR DIRECT ANALYSIS
Zijuan Zhang, Jiale Xing, Yaopeng Liu* and Guochang Li

321

Connections

SEISMIC PERFORMANCE OF THREE-DIMENSIONAL STEEL BEAM-COLUMN CONNECTIONS
Y.L. Xu*, Y.F. Shang and Y.X. Su

331

EXPERIMENTAL STUDY ON TRUSS TYPE STEEL REINFORCED CONCRETE JOINTS
T. Chen*, X.L. Gu, W.R. Fu, Q.H. Huang and B. Peng

340

EXPERIMENTAL INVESTIGATION ON THE STRUCTURAL BEHAVIOR OF CORRODED SELF-DRILLING SCREW CONNECTIONS IN COLD-FORMED STEEL STRUCTURES

351
ULTIMATE STRENGTH, DUCTILITY AND FAILURE MODE OF HIGH-STRENGTH FRICIONAL BOLTED JOINTS MADE OF HIGH STRENGTH STEEL
Z.C. Qin*, H. Moriyama, T. Yamaguchi, M. Shigeishi, Y. Xing and A. Hashimoto

EXPERIMENTAL STUDY ON BOLTED CONNECTIONS IN COLD-ROLLED ALUMINIUM PORTAL FRAMES
H.C. Nguyen and C.H. Pham*

EXPERIMENTAL STUDY ON BEHAVIOR OF THE GUSSET-PLATE JOINT OF ALUMINUM ALLOY PORTAL FRAME
J. Liu*, X.N. Guo and Y.F. Luo

PARAMETRIC STUDIES ON SCF DISTRIBUTION OF THREE-PLANAR TUBULAR Y-JOINTS UNDER IN-PLANE BENDING MOMENT
S.L. Bao*, Y.T. Tai, Y. Tian, X.Y. Zhao and R.N. Li

PARAMETRIC STUDIES ON THE MOMENT RESISTANT BEAM-COLUMN CONNECTION BEHAVIOR OF CONCRETE FILLED DOUBLE STEEL TUBULAR COLUMNS AND I STEEL BEAMS
M. Sulthana*, T. Supritha

LOAD TRANSFER MECHANISM OF STEEL GIRDER-RC PIER CONNECTION IN COMPOSITE RIGID-FRAME BRIDGE
H.X. Liu*, Xianlin Wang, Mao Feng Yu, Binqiang Guo and Yuqing Li

COMPARISON OF MECHANICAL BEHAVIOR BETWEEN LONGITUDINAL LAP-WELDED JOINTS AND TRANSVERSE FILLET WELDED JOINTS OF HIGH STRENGTH STEEL
S.H. Jiang, M.M. Ran*, F. Xiong and Y.C. Zhong

STUDY ON THE STATIC BEHAVIOR OF COLD-FORMED STEEL FABRICATED BEAM-COLUMN JOINT
L.P. Wang*, A. Abubakar B* and J. Li

NUMERICAL STUDY OF THE PRELOAD FORCE LOSS OF CORRODED HIGH-STRENGTH BOLTS
Y. Jin, X. Zhang and Z.Y. Kong*

Corrosion, Fracture & Collapse

ANTI-WIND CAPACITY CHECK AND COLLAPSES ANALYSIS OF EXISTING TRANSMISSION TOWER
W.T. Zhang*, Y.Q. Xiao, C. Li and Q.X. Zheng
DYNAMIC ANALYSIS OF LONG-SPAN TRANSMISSION TOWER-LINE SYSTEM UNDER DOWNBURST
D.K. Zhang*, H.Z. Deng, X.Y. Hu

APPLICATION RESEARCH OF V CONTAINING HIGH STRENGTH WEATHERING STEEL IN STEEL STRUCTURE BUILDING
Z.R. Li*, K.Y. Cui, C.W. Wang and Shu Chen

EFFECT OF VARIOUS BOUNDARY CONSTRAINTS ON THE COLLAPSE BEHAVIOR OF MULTI-STORY COMPOSITE FRAMES
Z. Tan, W.H. Zhang*, X.Y. Song, B. Meng, C.F. Li, and S.C. Duan

Design & Analysis

STRENGTHENING DESIGN AND MECHANICAL BEHAVIOR ANALYSIS OF THE MAIN STRUCTURE FOR AN INDUSTRIAL WORKSHOP WHEN EQUIPMENT CHANGED
B. Jiang*, L. Jiang, S.C. Sang, Y.Y. Li, Y.G. Wu

ENHANCEMENT OF ANTI-COLLAPSE CAPACITY OF STEEL FRAME WITH OPENINGS IN BEAM WEB
B. Meng*, W.H. Zhong and Jiping Hao

INNOVATION AND PRACTICE IN BUILDING STRUCTURE DESIGN
Y.Q. Zhang*, J.M. Ding and Z. Zhang

CORRELATION BETWEEN RANDOM LOCAL MECHANICAL PROPERTIES OF STRUCTURAL STEEL
A. Machowski, M. Maslak* and M. Pazdanowski

RESEARCH ON CALCULATION METHOD OF LOADED COMPRESSION MEMBER OF SINGLE-LIMB FIRE-CURVED EQUILATERAL DOUBLE SPLICING T-SHAPED ANGLE STEEL
X.D. Li*, Z.G. Fang, J.Q. Ye, D.H. Sun and W. Yao

ROTATIONAL STIFFNESS MODEL FOR SHALLOW EMBEDDED STEEL COLUMN BASES
X.X. Xu*, X.Z. Zhao and S. Yan

STUDY ON MECHANICAL PROPERTIES OF SIMPLIFIED STEEL FRAME MODEL WITH EXTERNAL WALL PANELS
Y.Z. Liu* and W.Y. Zhang

INTEGRATED DESIGN OPTIMIZATION FOR LONG SPAN STEEL TRANSFER TRUSS
AT REDEVELOPMENT OF HONG KONG KWONG WAH HOSPITAL
X.K. Zou, Y. Zhang, Y.P. Liu*, L.C. Shi and D. Kan

Direct Analysis

SECOND-ORDER DIRECT ANALYSIS FOR STEEL H-PILES ACCOUNTING FOR POST-DRIVING RESIDUAL STRESSES
W.H. Ouyang, L. Chen and S.W. Liu*

Fatigue

RECONSTRUCTION METHOD OF FATIGUE DAMAGE STATE OF IN-SERVICE STEEL BRIDGE WITHOUT LOAD INFORMATION
L.T. Da*, Q.H. ZHANG, M.Z. Li and C. Cui

FATIGUE PERFORMANCE OF RIB-TO-DECK JOINTS STRENGTHENED WITH INTERNAL WELDING
M.Z. Li*, Q.H. Zhang, J. Li, L.T. Da and C. Cui

EXPERIMENTAL INVESTIGATION ON RESIDUAL STRESS DISTRIBUTION AND RELAXATION EFFECT AT DOUBLE-SIDE WELDED RIB-TO-DECK JOINTS OF ORTHOTROPIC STEEL DECKS
Y. Ma*, C. Cui, Q.H. Zhang and W.L. Lao

FATIGUE BEHAVIOUR OF TITANIUM-CLAD BIMETALLIC STEEL PLATE WITH DIFFERENT INTERFACIAL CONDITIONS

MECHANICAL PROPERTIES AND SIMULATION METHOD OF STRUCTURAL STEEL AFTER HIGH CYCLE FATIGUE DAMAGE
Q. Si, Y. Ding, L. Zong* and H. Liu

EXPERIMENTAL STUDY ON WELDING RESIDUAL STRESS OF TWO-WAY STIFFENED STEEL PLATES
Z. Shao, Y.X. Li, S.Y. Song, W.L. Jin, Y.Q. Liu*

Volume II

Fire
BENDING MECHANICAL PROPERTIES OF STEEL - WELDED HOLLOW SPHERICAL JOINTS AT HIGH TEMPERATURES
L. Wang, H.B. Liu*, H. Dong, and X.N. Liu

HIGH STRENGTH STEEL BEAM BEHAVIOR UNDER FIRE EXPOSURE CONSIDERING CREEP
H. Al-azzani*, W.Y. Wang and A. Sharhan

EXPERIMENTAL INVESTIGATION ON MECHANICAL PROPERTIES OF GRADE 1670 STEEL WIRES AT AND AFTER ELEVATED TEMPERATURE

FINITE ELEMENT SIMULATION FOR ULTRA-HIGH-PERFORMANCE CONCRETE-FILLED DOUBLE-SKIN TUBES EXPOSED TO FIRE
A.H.A. Abdelrahman*, M. Ghannam, S. Lotfy, and M. AlHamaydeh

High-Strength Steel
EXPERIMENTAL INVESTIGATION OF RESIDUAL STRESS IN WELDED T-SECTION BY DOMESTIC Q460 HIGH STRENGTH S
X.L. Xiong*, F.R. Nkuichou, T. Wang, M. Ma and K. Du

CORROSION EFFECTS ON MECHANICAL PROPERTIES OF Q620 HIGH-STRENGTH STEEL
N. Wang, J.M. Hua, X.Y. Xue*, Q.Q. Huang, F. Wang

Impact and protection
TENSILE BEHAVIOR OF T-STUB SUBJECTED TO STATIC AND DYNAMIC LOADS
H. Huang, L.M. Ren, K. Chen, X.J. Li, L. Wang and B. Yang*

Intelligent Construction
APPLICATION OF HYDRAULIC SYNCHRONOUS LIFTING TECHNOLOGY IN CONSTRUCTION OF LONG-SPAN HYBRID STEEL STRUCTURES
M.L. Zhang*, W. Liu, Z. Lei, D.G. Wang, J.Y. Wang, L.Y. Zhou* and X.P. Shu

TESTING OF ADDITIVELY MANUFACTURED STAINLESS STEEL MATERIAL AND CROSS-SECTIONS
R.Z. Zhang*, L. Gardner and C. Buchanan

EMBODIED CARBON CALCULATION AND ASSESSMENT FOR STEEL STRUCTURE PROJECT
D. Chan, W. Sun and Y.Y. Wang*
COMPLETE SET CONSTRUCTION TECHNOLOGY OF LARGE OPENING CABLE DOME STRUCTURE BASED ON INTEGRATED Y.Y. Shang*, Z.S Xing, C.Q. Wu, F.S Lu and B. Luo

COMPLETE SET ROTATION-LIFTING CONSTRUCTION TECHNOLOGY FOR FREE-FORM SURFACE ROOF STRUCTURES WITH LARGE ELEVATION DIFFERENCE Z.S. Xing, S.R. Jia, Z.H. Zhang and D.C. Ye

New Materials

FINITE ELEMENT ANALYSIS ON BEHAVIOR OF HCFHST MIDDLE LONG COLUMNS WITH INNER I-SHAPED CFRP UNDER AXIAL LOAD Guochang Li, Runze Li* and Zhijian Yang

STUDY ON THE MECHANICAL BEHAVIOR OF GFRP PLATE-CONE CYLINDRICAL RETICULATED SHELL X. Wang, L. Chen, Y.H. Huang, F. Wang* and X. Zhang

EXPERIMENTAL STUDY ON MECHANICAL PROPERTIES AND OPTIMIZATION OF CHOPPED BASALT FIBER REINFORCED CONCRETE Q. Liu, Z.X. Yu and R. Guo*

STUDY ON MECHANICAL PROPERTIES OF STAINLESS STEEL PLATE SHEAR WALL STRENGTHENED BY CORRUGATED FRP Y.P. Du*and L. Zhong

DESIGN OF THE DEPLOYABLE-FOLDABLE ACTUATOR AND VIBRATION CONTROL DEVICE BASED ON THE SHAPE MEMORY ALLOYS WITH A TWO-WAY EFFECT Di SONG*, You-Jun LU, and Chang-Qing MIAO

Seismic Resistance

FEASIBILITY STUDY OF VISCOELASTIC HYBRID SELF-CENTERING BRACE (VSCB) FOR SEISMIC-RESISTANT STEEL FRAMES Y.W. Ping, C. Fang* and Y.Y. Chen

TEST ON RESILIENCE CAPACITY OF SELF-CENTERING BUCKLING RESTRAINED BRACE WITH DISC SPRINGS Y.K. Ding*, M. Soy, M.K. Tang and W.Y. Zhang

MECHANICAL PROPERTIES OF KINKED STEEL PLATES AND THEIR APPLICATIONS IN FRAME STRUCTURES X.J. Yang, F. Lin* and C.P. Liu
SEISMIC COLLAPSE AND DEBRIS DISTRIBUTION OF STEEL FRAME STRUCTURES WITH INFILL WALLS
Z. Xu and F. Lin*

ANALYSIS OF TRANSIENT STRUCTURAL RESPONSES OF STEEL FRAMES WITH NON-SYMMETRIC SECTIONS UNDER EARTHQUAKE MOTION
W.L. Gao, L. Chen and S.W. Liu*

SEISMIC RESILIENCE ASSESSMENT OF A SINGLE-LAYER RETICULATED DOME DURING CONSTRUCTION
T.L. Zhang and J.Y. Zhao*

Stability

LOCAL BUCKLING (WRINKLING) OF profiled METAL-FACED INSULATING SANDWICH PANELS - A PARAMETRIC STUDY
M.N. Tahir* and E. Hamed

COMPARATIVE STUDY ON STABILITY OF WELDED AND HOT-ROLLED Q420 L300×30 COLUMNS
A.P. Chou and G. Shi*

ELASTIC BUCKLING OF OUTSTAND STAINLESS-CLAD BIMETALLIC STEEL PLATES SUBJECTED TO UNIAXIAL COMPRESSION
Y.X. Mei* and H.Y. Ban

IMPERFECTION SENSITIVITY OF NON-TRIANGULATED CYLINDRICAL SHELL CONFIGURATIONS
R. Kolakkattol*, K.D. Tsavdaridis, and A.S. Jayachandran

Stainless Steel

MATERIAL PROPERTIES AND LOCAL STABILITY OF WAAM STAINLESS STEEL PLATES WITH DIFFERENT DEPOSITION RATES
S.I. Evans* and J. Wang

A REEXAMINATION ON CALIBRATION OF CYCLIC CONSTITUTIVE MODEL FOR STRUCTURAL STEELS

FINITE ELEMENT MODELING OF CONCRETE-FILLED STAINLESS-CLAD BIMETALLIC STEEL SQUARE TUBES UNDER AXIAL COMPRESSION
Z.J. Chen*, H.Y. Ban, Y.Q. Wang
Structure Systems

INVESTIGATION OF CYCLIC BEHAVIOR OF FULL-SCALE TREE-LIKE HOLLOW STRUCTURAL SECTION COLUMNS WITH INFILLED CONCRETE
D. Gan*, Z.H. He, and H.H. Huang

ANALYSIS OF THE SEISMIC BEHAVIOR OF INNOVATIVE ALUMINIUM ALLOY ENERGY DISSIPATION BRACES
B. Jia*, Q.L. Zhang and T. Wu

SHAKING TABLE TEST OF NEW LIGHT STEEL STRUCTURE SYSTEM

Testing & Monitoring

THE CRACK DETECTION METHOD OF LONGITUDINAL RIB BUTT WELD OF STEEL BRIDGE BASED ON ULTRASONIC LAMB WAVE
D.K. Zhang*, Q.H. Zhang, C. Cui and S.J. Qiu

ON FIELD-MEASURED VERTICAL TEMPERATURE GRADIENT OF BOX GIRDER IN STEEL BRIDGES
Zhiwen Zhu*, Tang Qin, Xiaowan Chen
Preface

These proceedings contain the papers presented at the TENTH INTERNATIONAL CONFERENCE ON ADVANCES IN STEEL STRUCTURES (ICASS 2020) held in Chengdu, China, from 21 to 23 August 2022. The international conference series on Advances in Steel Structures was initiated in 1996 under the support of The Hong Kong Polytechnic University, which remains very active in fostering its continuation - joined a few years later by the Hong Kong Institute of Steel Construction.

These proceedings bring together most recent findings in numerical, theoretical and experimental research, as well as its practical implementation in design practice in the areas of Assembled Structure, Bridge, Cold-formed Steel, Composite, Connections, Corrosion, Fracture & Collapse, Design & Analysis, Direct Analysis, Fatigue, Fire, High-Strength Steel, Impact and Protection, Intelligent Construction, New Material, Seismic Resistance, Stability, Stainless Steel, Structure Systems, Testing & Monitoring. The papers presented in these proceedings come from a wide range of countries/regions and will be a great reference source.

Specially, the subject matter has been categorized under the broad heading of:

**Volume I:** Keynotes Lectures, Assembled Structure, Bridge, Cold-Formed, Composite, Connections, Corrosion, Fracture & Collapse, Design & Analysis, Direct Analysis, Fatigue


Each of the papers was subjected to stringent review by a panel of experts in the respective area. This peer review began with an assessment of the submitted abstracts and following this, authors were invited to submit their full manuscripts. Each manuscript was then carefully reviewed by relevant experts, and their recommendations on accepting, rejecting or modifying the submissions were strictly adhered to, before inclusion in the conference proceedings.
Fire
BENDING MECHANICAL PROPERTIES OF STEEL–WELDED HOLLOW SPHERICAL JOINTS AT HIGH TEMPERATURES (ICASS 2020)

Lan Wang 1, Hongbo Liu 2*, Heng Dong 3, and Xiaona Liu 3

1 Tianjin fire research institute of MEM, Tianjin, China  
E–mails: wanglan@tfri.com.cn
2 State Key Laboratory of Hydraulic Engineering Simulation and Safety, Tianjin University, Tianjin, China  
E–mail: hbliu@tju.edu.cn
3 Department of Civil Engineering, Tianjin University, Tianjin, China  
E–mail: dhstrive@163.com, E–mail: liuxiaona96@163.com  
(*Corresponding Author)

Abstract: Spatial grid structure is a commonly used long–span structural form due to its various advantages, such as light weight, high strength, low construction cost, and simple construction. Steel–welded hollow spherical joints are widely applied in traditional spatial grid structures. Circular steel tube– and H–shaped steel–welded hollow spherical joints have been applied in practical engineering projects because of aesthetic appearance and structural stress requirements of modern spatial grid structures. Existing studies have mainly focused on the axial compression behaviors of steel–welded hollow spherical joints at high temperatures during fire disasters. However, few studies have discussed the bending mechanical properties of hollow and H–shaped steel–welded hollow spherical joints. This study conducted finite element analysis on the bending mechanical properties of circular steel tube– and H–shaped steel–welded hollow spherical joints at high temperatures. Influences of parameters including the sizes of welded hollow sphere and fashioned iron on the bending mechanical properties of welded hollow spherical joints were considered in the finite element analysis. Moreover, changes in the failure modes, flexural capacities, and flexural rigidities of circular steel tube– and H–shaped steel–welded hollow spherical joints with the increase in temperatures were analyzed. The fitting formulas of the ultimate bearing capacity and initial flexural rigidity of circular steel tube– and H–shaped steel–welded hollow spherical joints at high temperatures were proposed on the basis of parametric analysis.

Keywords: Circular steel tube–welded hollow spherical joint; H–shaped steel–welded hollow spherical joint; At high temperatures; Flexural capacity; Initial flexural rigidity

DOI: 10.18057/ICASS2020.P.146

1 INTRODUCTION

Spatial structure has reasonable stresses, unique structure, easy construction, and high economic efficiency; this structure can also solve the coverage problem of long–span architectural space. Spatial structure is becoming a vital development field. Many joints, including welded spherical [1–2], bolted spherical [3], hub–type [4], assembly [5], tubular [6] and cast steel [7] joints, have been developed successively in the development process of space structure. Among them, welded spherical joints are used the most [8].
Existing studies on welded hollow spherical joints have mainly focused on mechanical properties at room temperature, high temperatures, and after high temperatures. With respect to the mechanical properties of welded hollow spherical joints at room temperature, Han et al. calculated the ultimate bearing capacity of the welded hollow spherical joint based on a 3D degeneration curved shell finite element and proposed that the welded hollow spherical joint developed strength fracture under axial tensile loads and elastoplastic buckling failure under axial compressive loads. On the basis of regression analysis, they proposed a theoretical formula to calculate the bearing capacity of the welded hollow spherical joint and the theoretical and practical calculation formulas of the bearing capacity of the joint under eccentric loads. Dong et al. \cite{12-16} experimentally studied typical circular steel tube–, square steel tube–, and rectangular steel–welded hollow spherical joints and proposed a calculation method of the bearing capacity of joints with considerations to the axial force and bending moment of three rod forms and their interaction. Liu et al. \cite{17} concluded the calculation formula of the axial compressive and tensile strengths of H–shaped steel–welded hollow spherical joints through experiments and finite element analysis. Zhang \cite{18} proposed the calculation formulas of the bearing capacities and rigidities of H–shaped steel–welded hollow spherical joints under axial compressive and pure bending loads and analyzed the degeneration laws of joint rigidity. Zhao et al. \cite{19-20} analyzed two corrosion forms through nonlinear numerical analysis and discussed the influences of corrosion on the mechanical properties of welded hollow spherical joints under simple axial force and interaction of axial force and bending moment. For welded hollow spherical joints at and after high temperatures, existing studies have mainly focused on hollow spherical joints welded with circular steel tube. Xue \cite{21} conducted experimental studies and numerical simulation on the compressive mechanical properties of joints at high temperatures. Wang and Liu et al. \cite{1} \cite{22-24} performed experimental study and numerical simulation of joints under axial compressive and biased loads after high temperatures.

In summary, scholars in China and foreign countries have conducted relatively systematic studies on the mechanical properties of welded hollow spherical joints in traditional forms at room temperature, including stress mechanism, bearing capacity, rigidity characteristics, and design method of the joints under axial stress and bending moment. However, few studies have been performed on the mechanical degradation laws of joints under bending stress and high temperature. The current study explored the flexural capacities and rigidities of circular steel tube– and H–shaped steel–welded hollow spherical joints at high temperatures.

2 NUMERICAL SIMULATION

2.1 Conditional hypotheses

2.1.1 Mechanical model

The values of bending moment and corner were defined in this study to obtain the accurate bending moment–corner curve of spherical joints. The 1/4–span symmetric loading mode was applied (Fig. 1). The total node length is $a$. Thus, the distance from the loading point to the end is $a/4$, and the bending moment that the middle spherical joint bears is $M = Fa/4$. During the loading process, a compressive deformation is developed at point A as the connection between the steel tube and hollow sphere, and a tensile deformation is developed at point B. The displacements at points A and B along the length direction of rod piece are $\Delta A$ and $\Delta B$, respectively. Therefore, the corner at the connection is $\theta = |\Delta A - \Delta B|/h$ (radian system), where $h$ is the sectional height of the jointing element.
2.1.2 Constitutive model of steel material at high temperatures

The mechanical properties of the material, such as elasticity modulus, yield strength, and ultimate strength, may change under high temperatures. Analysis accuracy of structures at high temperatures is mainly determined by the stress-strain relation of the used material at high temperatures, which refers to determination of various mechanical indexes. Eurocode 3\textsuperscript{[25]} regulated that the smooth curve model of the stress strengthening of steel materials was ignored at high temperatures during fire disasters (Fig. 2). The reduction factors of the yielding strength and initial elasticity modulus of steel material at different temperatures are listed in Table 1. The Poisson’s ratio of steel material was influenced slightly by high temperatures. Thus, the Poisson’s ratio of steel material at high temperatures during fire disasters was set equal to that at room temperature with a value of 0.3.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{fig2.png}
\caption{Neglected strengthening smooth curve model.}
\end{figure}

\begin{table}[h]
\centering
\caption{Reduction factors of the mechanical properties of steel material at high temperatures.}
\begin{tabular}{|c|c|c|}
\hline
Temperatures $T/\degree C$ & Reduction factors of yield strength & Reduction factors of initial elasticity modulus \\
\hline
20 & 1.000 & 1.000 \\
100 & 1.000 & 1.000 \\
200 & 1.000 & 0.900 \\
300 & 1.000 & 0.800 \\
400 & 1.000 & 0.700 \\
500 & 0.780 & 0.600 \\
600 & 0.470 & 0.310 \\
700 & 0.230 & 0.130 \\
800 & 0.110 & 0.090 \\
900 & 0.060 & 0.0675 \\
1000 & 0.040 & 0.0450 \\
1100 & 0.020 & 0.0225 \\
1200 & 0.000 & 0.000 \\
\hline
\end{tabular}
\end{table}
2.1.3 Material attributes at high temperatures

Elasticity modulus and thermal conductivity at different temperatures are listed in Table 2.

<table>
<thead>
<tr>
<th>Temperatures /°C</th>
<th>20</th>
<th>100</th>
<th>200</th>
<th>300</th>
<th>400</th>
<th>500</th>
<th>600</th>
<th>700</th>
<th>800</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elasticity modulus /GPa</td>
<td>206.0</td>
<td>206.0</td>
<td>185.4</td>
<td>164.8</td>
<td>144.2</td>
<td>123.6</td>
<td>63.9</td>
<td>26.8</td>
<td>18.5</td>
</tr>
<tr>
<td>Thermal conductivity W/m·°C</td>
<td>44.24</td>
<td>41.58</td>
<td>38.25</td>
<td>34.92</td>
<td>31.59</td>
<td>28.26</td>
<td>24.93</td>
<td>21.60</td>
<td>18.27</td>
</tr>
</tbody>
</table>

Table 2: Elasticity modulus and thermal conductivity at different temperatures.

Q345 steel was applied in all numerical simulations. At present, norms of different countries propose regulations over the coefficient of thermal expansion. In general, the coefficient of thermal expansion is unrelated to temperatures and is set to 1.4×10⁻⁵ m/(m·°C).

2.2 Finite element model of steel–welded hollow spherical joints and simulation results

2.2.1 Finite element model

Circular steel tubes were connected with a hollow sphere tightly through welded joints. In the process of modeling, the welding between the section of the circular steel tube and the external surface of hollow sphere was simulated by setting TIE constraint. The reference points were set at two ends of the steel tubes. The coupling constrains were set between each pair of the reference point and terminal section of the steel tubes. The degree of freedom (DOF) of translation at one reference point along x, y, and z directions was restrained, while the DOF of translation along the x and z directions and the DOF of rotation along the y direction at the other reference point were restrained. The loading end was at 1/4 of midspan of the model, above which a reference point was set. This reference point coupled with the steel tube section at the corresponding position. The vertical displacement load was applied on the joints through the reference point. The whole model considered influences of geometric nonlinearity. The unit type was C3D8R, which was an 8–joint linear hexahedral unit. The meshing density of all models was set to 5 mm, and the meshing density at the flange–web boundary and spherical thickness was changed to 4 mm. The overall finite element model of steel tube–welded spherical joints is shown in Fig. 3.

With references to construction requirements of the joints in Space Grid Technical Regulations, considerations were given to the diameter and wall thickness of the welded hollow sphere, steel tube diameter, and so on. A total of 69 models of steel–welded hollow spherical joints (20 models at room temperature and 49 models at high temperatures) were designed. According to relevant studies, the wall thickness of steel tubes influenced the bearing capacity of the joints slightly. In this model analysis, the influences of the wall thickness of steel tube were neglected, and the wall thickness of the steel tube was set equal to that of the hollow sphere.

Fig. 3 Finite element model of steel–welded hollow spherical joints.
2.2.2 Finite element simulation results

The different specifications of hollow spherical joints welded with circular steel tubes develop similar failure modes under bending moment (Fig. 4). Under bending moment, positions with maximum stress concentrate at the welding joints between steel tubes and sphere, and these positions reach the yield strength first. The upper part of the joint mainly bears compressive stress, while the lower part mainly bears tensile stress. With the increase in loads, the plastic region at the steel tube spherical boundary expands around the hollow sphere, and stress on the steel tube begins to increase at a slow rate. The upper part of the joint between the hollow spherical joint and steel tube develops serious compressive invaginations, and the joint reaches the ultimate bearing capacity. Under this circumstance, the steel tube stress level is very low. The steel–welded hollow spherical joints show similar failure modes at high temperatures.

Fig. 4 Typical failure mode of steel–welded hollow spherical joints under bending moment.

2.3 Finite element model of H–shaped steel–welded hollow spherical joints and simulation results

2.3.1 Finite element model

The models applied solid modeling. The simulation unit type, meshing density, simulation parameters, loading mode, and loading position were same as those in the bending simulation of steel–welded hollow spherical joints. The overall finite element model of H–shaped steel–welded hollow spherical joints is shown in Fig. 5. A total of 210 H–shaped steel–welded hollow spherical joint models (34 models at room temperature and 176 models at high temperatures) were designed with considerations to diameter of welded hollow sphere (D), wall thickness of welded hollow sphere (t), H–shaped steel sectional height (h), H–shaped steel section width (b), H–shaped steel sectional flange width (tf), and H–shaped steel sectional web width (tw). Eight temperatures in the range of 20 °C–800 °C were set in each joint model.

Fig. 5 H–shaped steel–welded hollow spherical joint model.

2.3.2 Finite element simulation results

The failure modes of different specifications of H–shaped steel–welded hollow spherical joints under bending moment are similar according to the analysis results (Fig. 6). In the elasticity segment, the boundaries of H–shaped steel and spherical joint restrict mutually, and the stress increases quickly. The boundary between the H–shaped steel and spherical joint is the first position that reaches plasticity, and the fashioned iron is generally at a low stress level.
With the increase in loads, the plasticity region at the boundary between the fashioned iron and spherical joint expands around the hollow sphere, and the stress on the flange of the fashioned iron begins to increase. However, the growth rate of stress is relatively low, and stress on the web increases the most but slowly. The upper part of the joint between the hollow sphere and fashioned iron develops invagination and can no longer bear loads. In this case, relatively low stress levels are observed at the flange of the fashioned iron and web.

Fig. 6 Typical failure mode of H–shaped steel–welded hollow spherical joints under bending moment.

3 RESULTS ANALYSIS

3.1 Analysis of the bending mechanical properties of steel–welded hollow spherical joints

3.1.1 Flexural capacity

The Space Grid Structure Technical Regulations (JGJ 7–2010) provided the calculation formula of ultimate bearing capacity of welded hollow spherical joints under axial pressure. Similarly, to simplify the calculation method, the calculation formula of bearing capacity of welded hollow spherical joints under bending moment can use Eq. (1):

\[ M = (A + B \frac{d}{D})t d^2 f, \]

where \( A \) and \( B \) are the coefficients that have to be determined, \( t \) is the wall thickness of the welded hollow sphere, \( d \) is the diameter of steel tube, \( D \) is the diameter of welded hollow sphere, and \( f \) is the tensile strength standard of steel materials.

The bearing capacities obtained by finite element simulation and calculated from the theoretical formula were compared and analyzed. \( d/D \) was used as the \( x \)-axis, and the dimensionless parameter \( M/t d^2 f \) was used as the \( y \)-axis. Analysis data are drawn in Fig. 7. To assure the relative safety of the calculation formula, the lower envelope line of all finite element results was chosen (solid line in Fig. 7). Clearly, \( A=0.06 \) and \( B=0.87 \). In other words, the solid line equation is \( 0.06+0.87 \frac{d}{D} \).

In summary, the practical calculation formula of the bearing capacity of welded hollow spherical joints under bending moment can be expressed as

\[ M = (0.06 + 0.87 \frac{d}{D})t d^2 f. \]
3.1.2 Flexural rigidity

Through numerical analysis, key attentions were paid to influences of parameters, such as tube diameter–spherical diameter ratio \( (d/D) \) and wall thickness of sphere \( (t) \), on the flexural rigidity of steel–welded hollow spherical joints. Relevant influencing laws are shown in Fig. 8. The comparison shows that \( d/D \) and \( t \) are primary influencing factors of flexural rigidity. With the increases in \( d/D \) and \( t \), the flexural rigidity of welded spherical joints increases significantly. Therefore, the two parameters are significantly positively correlated with the flexural rigidity of welded spherical joints.

The influences of \( d/D \) and \( t \) on the flexural rigidity of steel–welded hollow spherical joints were considered to obtain an accurate calculation formula of flexural rigidity. The formula of flexural rigidity was fitted on the basis of the finite element analysis results. The calculation formula of flexural rigidity is obtained as

\[
K=2\pi E\left(0.0421\frac{t d^3}{D} + 14.76\frac{t^3 d^3}{D^3}\right).
\]  

(3)

The finite element analysis results were compared with the calculated results from Eq. (3). The average error is 4.73%. Thus, Eq. (3) can be used to calculate the flexural rigidity of steel–welded hollow spherical joints under bending moment accurately.
3.2 Analysis of the bending mechanical properties of H–shaped steel–welded spherical joints

3.2.1 Flexural capacity

A dimensionless parameter \( \lambda_M = \frac{M_p}{f_u (h + b + t_f)} \) of the H–shaped steel–welded hollow spherical joints was introduced to analyze the influences of different factors. The results show that \( D/t, h/D, \) and \( b/D \) exhibit similar influencing laws on \( \lambda_M = \frac{M_p}{f_u (h + b + t_f)} \).

\[
M_u = (0.151 - 0.00167 \frac{D}{t} + 0.0933 \frac{h}{D} + 0.4 \frac{b}{D})(h + b + t_f) \cdot f_u
\]

where 200 mm \( \leq D \leq 400 \) mm, 20 \( \leq D/t \leq 50 \), 0.35 \( \leq h/D \leq 0.7 \), and 0.35 \( \leq b/D \leq 0.7 \). \( f_u \) is the ultimate strength value of steel material.

The maximum error of the ultimate flexural capacity of H–shaped steel–welded hollow spherical joints between the finite element simulation and calculated results is \(-7.01\%\), and the maximum error is \(1.97\%\). Therefore, Eq. (4) can reflect the real results well.

3.2.2 Flexural rigidity

The parametric analysis indicates that the initial flexural rigidity of H–shaped steel–welded sphere is related to the diameter and wall thickness of the welded sphere and the height, width, and flange thickness of the fashioned iron. With considerations to the influences of these parameters, a change coefficient of compound sectional moment of inertia is introduced in

\[
\beta = x_1 + x_2 \frac{D}{t} + x_3 \frac{h}{D} + x_4 \frac{b}{D} + x_5 \frac{t_f}{D}
\]

The simplified formula of H–shaped steel sectional moment of inertia is

\[
I_h = \frac{1}{12} t_w h^3 + \frac{1}{2} h^2 t_f (b - t_w)
\]

Therefore, the initial flexural rigidity of welded spherical joints can be expressed as

\[
K_K = \frac{EI_h}{D} = \frac{E \left( \frac{1}{12} t_w h^3 + \frac{1}{2} h^2 t_f (b - t_w) \right)}{D(x_1 + x_2 \frac{D}{t} + x_3 \frac{h}{D} + x_4 \frac{b}{D} + x_5 \frac{t_f}{D})}
\]

In accordance with the finite element fitting results, the calculation formula of the initial flexural rigidity of H–shaped steel–welded hollow spherical joints can be gained as

\[
K_K = \frac{EI_h}{D} = \frac{E \left( \frac{1}{12} t_w h^3 + \frac{1}{2} h^2 t_f (b - t_w) \right)}{D(1.322 + 0.0433 \frac{D}{t} - 1.139 \frac{h}{D} - 0.257 \frac{b}{D} - 0.0343 \frac{t_f}{D})}
\]

The average error between the finite element simulation results of the initial flexural rigidity of H–shaped steel–welded hollow spherical joints and the calculated results is \(4.8\%\). This result indicates that Eq. (8) can reflect real results well.
3.3 Degradation laws of the bending mechanical properties of steel–welded hollow spherical joints at high temperatures

3.3.1 Degradation law of flexural capacity

To obtain the relationship between the ultimate flexural capacity of steel–welded hollow spherical joints and temperatures, the reduction factors of ultimate flexural capacity at high temperatures are defined as follows:

$$\eta_{u,T} = \frac{M_{u,T}}{M_{u,20}}$$

where $M_{u,T}$ is the ultimate flexural capacity of steel–welded hollow spherical joints when the temperature is $T$ and $M_{u,20}$ is the ultimate flexural capacity of steel–welded hollow spherical joints when the temperature is 20 °C.

The variation trend of the reduction factors of ultimate flexural capacity is shown in Fig. 9. As observed, the reduction factors of ultimate flexural capacity are related to temperatures. The reduction factors of the bearing capacity of components with different specifications show the same variation trend at high temperatures, which indicates that bearing capacity is less related to the section of components. The reduction factor of ultimate flexural capacity is approximately 0.916 at 200 °C, and it decreases continuously to around 0.876, 0.75, 0.601, 0.362, 0.19, and 0.103 at 300 °C, 400 °C, 500 °C, 600 °C, 700 °C, and 800 °C, respectively. Therefore, the steel–welded hollow spherical joints lose flexural capacity when the temperature exceeds 600 °C.

A multiple linear regression analysis on the reduction factors of the ultimate flexural capacity of steel–welded hollow spherical joints at high temperatures was conducted using the Origin software. Thus, the practical calculation formula is obtained, as shown in Eq. (10), where $T$ is the ambient temperature of steel–welded hollow spherical joints. The fitting diagram of the reduction factors of ultimate flexural capacity is shown in Fig. 10.

$$\eta_{u,T} = 1.0238 - 1.33 \times 10^{-3}T + 7.4686 \times 10^{-6}T^2 - 1.985 \times 10^{-8}T^3$$

$$20^\circ C \leq T \leq 800^\circ C.$$  

(10)

Fig. 9 Variation trend of reduction factors of ultimate flexural capacity.

Fig. 10 Fitting diagram of reduction factors of ultimate flexural capacity.
3.3.2 Degeneration law of initial flexural rigidity

To disclose the relationship between the initial flexural rigidity of steel–welded hollow spherical joints and temperatures, the reduction factors of initial flexural rigidity of steel–welded hollow spherical joints are defined as follows:

\[ \eta_{E,T} = \frac{K_{E,T}}{K_{E,20}}, \]  

(11)

where \( K_{E,T} \) is the initial flexural rigidity of steel–welded hollow spherical joints when the temperature is \( T \) and \( K_{E,20} \) is the initial flexural rigidity of steel–welded hollow spherical joints when the temperature is 20 °C.

The variation trend of the reduction factors of initial flexural rigidity is shown in Fig. 11. The reduction factors of initial flexural rigidity present significant variation laws with temperatures, which indicates a very low dispersion of data. However, the reduction factors of initial flexural rigidity have no significant relation with the size of the joints. The reduction factor of initial flexural rigidity is approximately 0.892 at 200 °C, and it decreases continuously to nearly 0.778, 0.649, 0.559, 0.291, 0.123, and 0.083 at 300 °C, 400 °C, 500 °C, 600 °C, 700 °C and 800 °C, respectively. In summary, the initial flexural rigidity at 500 °C decreases to around 55% of that at room temperature, and it further decreases to 29% of that at room temperature when the temperature is higher than 600 °C. Thus, high temperatures can cause loss of rigidity of the steel–welded hollow spherical joints.

A multiple linear regression analysis on the reduction factors of the initial flexural rigidity of steel–welded hollow spherical joints at high temperatures was conducted using the Origin software. As a result, the practical calculation formula is obtained, as shown in Eq. (12), where \( T \) is the ambient temperature of steel–welded hollow spherical joints.

\[ \eta_{E,T} = 0.99646 \times 10^{-4} T - 3.73 \times 10^{-6} T^3 + 2.4786 \times 10^{-9} T^4 \quad 20^\circ C \leq T \leq 800^\circ C \]  

(12)

3.4 Degeneration laws of the bending mechanical properties of H–shaped steel–welded hollow spherical joints at high temperatures

3.4.1 Degeneration laws of flexural capacity

To obtain the relationship between the ultimate flexural capacity of H–shaped steel–welded hollow spherical joints and temperatures, the reduction factors of ultimate flexural capacity at
The variation trend of the reduction factors of ultimate flexural capacity is shown in Fig. 13. As observed, the reduction factors of ultimate flexural capacity are closely related to temperature, but these factors are less related to the sections of components. The reduction factor of ultimate flexural capacity is approximately 0.923 at 200 °C, and it decreases continuously to nearly 0.882, 0.759, 0.604, 0.362, 0.181, and 0.096 at 300 °C, 400 °C, 500 °C, 600 °C, 700 °C, and over 800 °C, respectively. Therefore, the H–shaped steel–welded hollow spherical joints lose flexural capacity when the temperature exceeds 600 °C. On the contrary, the reduction amplitude of ultimate flexural capacity at high temperatures is smaller than that of plastic flexural capacity, and the welded spherical joints that are designed according to plastic flexural capacity at high temperatures are safer.

A multiple linear regression analysis on the reduction factors of the ultimate flexural capacity of H–shaped steel–welded hollow spherical joints at high temperatures was performed using the Origin software. Accordingly, the practical calculation formula is obtained, as shown in Eq. (13), where $T$ is the ambient temperature of the H–shaped steel–welded hollow spherical joints.

$$\eta_f = 1.0239 - 0.00135T + 7.846 \times 10^{-6}T^2 - 2.084 \times 10^{-4}T^3 + 1.4157 \times 10^{-11}T^4 \quad 20^\circ C \leq T \leq 800^\circ C$$

(13)

3.4.2 Degradation law of initial flexural rigidity

To disclose the relationship between the initial flexural rigidity of H–shaped steel–welded hollow spherical joints and temperatures, the reduction factors of initial flexural rigidity were defined with reference to the analysis method of steel–welded hollow sphere.

The variation trend of the reduction factors of initial flexural rigidity is shown in Fig. 15. The reduction factors of initial flexural rigidity present significant variation laws with temperatures, which indicates a very low dispersion of data. However, the reduction factors of initial flexural rigidity have no significant relation with the size of the joints. The reduction factor of initial flexural rigidity is approximately 0.903 at 200 °C, and it decreases continuously to nearly 0.803, 0.702, 0.601, 0.312, 0.126, and 0.087 at 300 °C, 400 °C, 500 °C, 600 °C, 700 °C, and 800 °C, respectively. In summary, the initial flexural rigidity at 500 °C decreases to around 60% of that at room temperature, and it further decreases to 31% of that at room temperature when the temperature is higher than 600 °C. Thus, high temperatures can cause loss of rigidity of the H–shaped steel–welded hollow spherical joints.
A multiple linear regression analysis on the reduction factors of the initial flexural rigidity of the H–shaped steel–welded hollow spherical joints at high temperatures was performed using the Origin software. Accordingly, the practical calculation formula is obtained, as shown in Eq. (14), where $T$ is the ambient temperature of steel–welded hollow spherical joints. The fitting diagram of the reduction factors of initial flexural rigidity is shown in Fig. 16.

\[
\eta_{k,T} = \begin{cases} 
1.012 - 4.563 \times 10^{-4}T - 7.483 \times 10^{-7} \times T^2 & 20^\circ C \leq T \leq 500^\circ C \\
3.948 - 0.00982T + 6.237 \times 10^{-6}T^2 & 500^\circ C \leq T \leq 800^\circ C 
\end{cases}
\] (14)

4 CONCLUSIONS

This study conducted numerical simulations on the bending mechanical properties of steel–welded and H–shaped steel–welded hollow spherical joints at high temperatures. The calculation formulas of the flexural capacity and initial flexural rigidity of the two kinds of joints were proposed on the basis of parametric analysis. Some major conclusions could be drawn as follows:

1) Stresses on the circular steel tube and the boundary between H–shaped steel and spherical joint increase quickly under pure bending load. The two positions reach plasticity first. Subsequently, the plastic region expands around the sphere from the boundary. During a failure, spherical joints develop significant invagination and show a buckling failure. However, most regions of the steel tube, flange, and web have not reached the ultimate strength.

2) Using empirical formulas and through parametric analysis, the influencing factors of the flexural capacity and initial flexural rigidity of steel–welded hollow spherical joints at room temperature and their calculation formulas are obtained. The degradation law of the bending mechanical properties of steel–welded hollow spherical joints at high temperatures is analyzed. In conclusion, the reduction factors of the flexural capacity and rigidity of steel–welded hollow spherical joints are mainly related to temperatures. Meanwhile, the calculation formulas of reduction coefficients are obtained.

3) Using empirical formulas and through parametric analysis, the influencing factors of the flexural capacity and initial flexural rigidity of H–shaped steel–welded hollow spherical joints at room temperature and their calculation formulas are acquired. The degradation law of the bending mechanical properties of H–shaped steel–welded hollow spherical joints at high temperatures is analyzed. In conclusion, the reduction factors of the flexural capacity and
rigidity of H–shaped steel–welded hollow spherical joints are mainly related to temperatures. Meanwhile, the calculation formulas of reduction coefficients are derived.

REFERENCES


HIGH STRENGTH STEEL BEAM BEHAVIOR UNDER FIRE EXPOSURE CONSIDERING CREEP
(ICASS’2020)

Hisham Al-azzani 1*, Weiyong Wang 1,2, Ahmed Sharhan 1

1 College of Civil Engineering, Chongqing University, Chongqing 400045, China.
2 Key Laboratory of New Technology for Construction of Cities in Mountain Area (Ministry of Education)
E-mails: hisham.alazzani@gmail.com, wywang@cqu.edu.cn
faressharhn@gmail.com

Abstract: The creep effect has been neglected in most previously undertaken research on fire resistance of restrained steel beams due to the absence of applicable creep models. A finite element model (FEM) was determined in this research to study fire resistance and the behavior of the restrained high-strength Q690 steel beams under fire exposure considering the high-temperature Fields&Fields creep model. Comparing the results obtained by the FEM with previous test results proved the validity of the FEM. Furthermore, a second FEM without a creep model was established to study the influence of creep on the fire resistance of restrained high-strength Q690 steel beams. Results showed that creep has a serious impact on the fire resistance of restrained high-strength Q690 steel-beams. Thus, ignoring creep will possibly lead to unsafe designs. Additionally, based on the results of the FEM, including creep effect, a simplified calculation method for restrained high-strength Q690 steel beams is presented to calculate the moment capacity. This calculation method is suitable for computing the critical temperature of restrained high-strength Q690 steel beams.

Keywords: Creep; Fire exposure; High strength steel; Restrained steel beams

DOI: 10.18057/ICASS2020.P.195

1 INTRODUCTION

In the past few years, restrained steel beams under fire loads have been widely investigated because of their realistic behavior due to rotational and axial restraints from the surrounding structures. Li and Guo [1] conducted two experiments on restrained steel beams to observe the difference in behavior at high temperatures between the two beams with different rotational and axial stiffness values. The comparison result showed that the stiffness of rotational and axial restraints has a significant role on the behavior of restrained steel beams in fire. Liu et al. [2] conducted a group of experiments to investigate the influence of axial restraints on steel beams under fire exposure. The test detected that the catenary action is mainly affected by axial restraints, which prevent the rapid increase in beam deflection.

The use of high-strength steels worldwide has become common because it is comparatively stronger compared with other average mild steels, especially those used for long-span structures and high-rise buildings [3,4]. Given that modern engineering applications require sufficient materials to meet the desired needs, Q690 steel is one of the mainly used high-strength steel to meet those requirements. Q690 steel is comparable to S690 EN 10025:2004 European structural steel and ASTM A514 steel in North America. Moreover, Q690 steel has a different chemical
composition from other mild steels. Therefore, this steel behaves differently by rapidly deteriorating under high temperatures. Wang et al. [5] conducted tensile tests on high-strength Q690 steel at different temperature scenarios and obtained values of elastic modulus and yield strength at each temperature. Their results showed a severe difference in the reduction factor between Q690 and mild steel.

Creep behavior occurs when a material can permanently deform under constant load or stress for a long period regardless of whether these stresses have not reached the yield strength of the material. Creep effect is a time-dependent deformation that mostly occurs at high temperatures. Consequently, material increases in length with time, leading to unsafe design if used. Many studies have been conducted to further understand this important phenomenon. Wang et al. [6] performed a group of numerical studies on restrained high-strength Q460 steel beams at high temperatures considering creep and found that the creep has a noticeable effect on the behavior of restrained high-strength-steel-beams in fire. Similarly, Kodur and Dwaikat [7] performed a group of numerical studies on restrained mild steel beams considering creep effect at high temperatures. Their results, which were compared with experimental results, also showed that the creep has a significant impact on the fire response of the restrained mild steel beams.

Almost all design codes disregarded creep effect addition. Thus, these codes do not provide an efficient method for estimating the behavior of restrained high-strength-steel-beams in fire. Consequently, many researchers are currently studying the effect of creep at high temperatures because of its importance in evaluating designs and providing comparable results to real conditions. Wang et al. [5,8,9] conducted a series of comprehensive tests to study creep deformation for different types of high-strength Q460 and Q690 steels and low alloy Q345 steel at different high temperatures. Moreover, Wang et al. proposed a creep model for each steel type. However, despite increasing efforts to understand creep behavior at high temperatures, information regarding the effects of creep behavior at high temperatures is limited. Therefore, considering the creep effect in the structure design codes is difficult. Additional research must be undertaken to build upon former works, such as that of Wang et al. [6], to gain a detailed understanding of the creep effect. Consequently, a numerical study was conducted and a method was proposed to assess the creep behavior and its influence on the fire response of restrained high-strength Q690 steel-beams. The novelty of the method lies in the consideration of creep and its simplicity and convenience, as described in this paper.

2 MATERIAL PROPERTIES

2.1 Mechanical properties of Q690 steel at high temperature

A considerable variance in the response of high-strength Q690 steel and other types of mild steel to fire loads is observed. This variance is due to the difference in the components and compositions of the chemical elements and that in the method of manufacture and fabrication process. Consequently, the reduction factors in Eurocode 3 cannot determine the correct mechanical properties of Q690 steel at different temperatures. Thus, the derived equations by Wang et al. [5] are selected for high-strength Q690 steel to determine the reduction factors at given temperatures.

\[
\frac{f_{y,T}}{f_y} = \frac{1}{1 + (T/538)^{10}}, \tag{1}
\]

\[
\frac{E_T}{E} = \frac{1}{1 + (T/534)^{5.5}}, \tag{2}
\]
where $f_{y,T}$ is the yield strength of Q690 steel at a given temperature; $f_y$ is the yield strength of Q690 steel at room temperature; $E_{T}$ is the elastic modulus of Q690 steel at a given temperature; $E$ is the elastic modulus of strength of Q690 steel at room temperature; $T$ is the temperature in Q690 steel.

Proposed equations by Poh [10] were selected to obtain the stress–strain relationship of high-strength Q690 steel. Poisson’s ratio is equal to 0.3.

2.2 Creep model

A few creep models, namely Fields&Fields [11], ANSYS [12], and Harmathy [13] creep models, have been used in fire resistance analysis due to the absence of test data, difficulty in conducting experiments, and numerous types of structural steel for construction. These models are based on creep tests for ASTM-A36 structural steel or Australian A149. As previously mentioned, the difference between steel types will lead to different behaviors at high temperatures and varying creep behavior. Wang et al. [5] proposed the creep power law for Q690. This creep model used by Abaqus finite element software package is known as Fields&Fields creep model as shown Fig. 1, which can be expressed as the Norton–Bailey equation as follows:

$$\varepsilon_{cr} = at^b(\sigma/d)^c,$$

(3)

where $\varepsilon_{cr}$ is the creep strain rate in steel; $t$ is time in (minutes); $\sigma$ is stress in (ksi); $a$, $b$, $c$ and $d$ are temperature-dependent material properties.

![Fig. 1 Proposed model for Fields&Fields creep model at 550°C [5].](image)

3  FINITE ELEMENT MODELING AND MODEL VALIDATION

3.1 Finite element model

Finite element software Abaqus [14] was chosen to perform the restrained high-strength Q690 steel beam model, which is suitable for simulating non-linear structural analysis, including heat transfer. The restrained Q690 steel beam was established in Abaqus with a uniform distributed load, as shown in Fig. 2. A solid element, which is capable of large time-dependent deformation at high temperatures, was selected to model the steel beams. Boundary conditions were selected as a SPRING element at the two ends of the steel beams to simulate the rotational and axial restraints. For simplicity, the quasi-static (VISCO) analysis type was
chosen to simulate creep at high temperatures and pre-defined field was used to apply temperatures on the steel beam.

Another model was made at room temperature to obtain the initial buckling mode, which was scaled to 1% beam length. Initial residual stresses were considered, and the initial values were taken from a study conducted by Zhang [15] for Q690 steel-welded H-section.

### 3.2 Model validation

An experimental investigation performed by Li and Guo [16] on restrained steel beams was chosen to validate the current established model. Two restrained steel beams with different rotational and axial stiffness were exposed to fire, both steel beams are H250×250×8×12, 4.5 m length, with two loading points (each 130 kN) were applied to estimate real conditions. Steel beams comprising Q235B steel have average yield strength of 291 MPa. The rotational and axial stiffness of the beams are $1.09 \times 10^8$ Nm/rad and 39540 N/mm, respectively. Luecke et al. [17] proposed to adjust the stress when calculating the creep strain. The stress adjustment factors are the ratios of the steel yield strength used in the creep model to that when the creep strain is calculated. The parameters of the Fields&Fields creep model were obtained through the fitting test data from [9] on Q460 steel. After stress adjustment, a high steel yield strength leads to a small creep under comparable stress conditions. The modified creep model can be expressed as follows:

$$
\varepsilon_{cr} = at^b \left( \frac{\sigma}{291} \right)^c.
$$

In the analysis, the following components are set at room temperature: the yield strength of Q235B steel is $f_y = 291$ MPa, the elastic modulus is $E_0 = 206 \times 10^3$ MPa, the coefficient of thermal expansion is chosen to be a constant $\alpha_s = 1.4 \times 10^{-5}$, and the Poisson ratio is 0.3. The temperature values of the steel beams were experimentally measured. The cross-section temperature is divided into the following two components for simplicity of analysis due to the large temperature difference between the measured points along the steel beam: 1) web and lower flange temperature is the average value of their experimentally measured temperature; 2) upper flange temperature is the average value of their experimentally measured temperature.
Fig. 3 Comparison between the experiment [1] and Abaqus FEM of Q235 restrained steel beam.

Fig. 3 shows that the Abaqus FEM has a good agreement with experiment test results considering creep in the analysis. Thus, neglecting the creep effect properly leads to inaccurate results during the calculation of deflections and thermal axial forces.

4 FIRE RESISTANCE DESIGN METHOD OF RESTRAINED Q690 STEEL BEAMS

Determining axial forces and bending moments of restrained steel beams considering creep under fire conditions is difficult. The restrained steel beams can continuously be loaded under large deflections due to the catenary effect. For the restrained Q690 steel-beams, the critical temperatures were taken as the critical temperatures $T_{cr}$ of the restrained Q690 steel-beams for fire resistance design for simplicity. The parametric analysis shows that the heating rate, span-to-depth ratio, rotational restraint stiffness, and cross-section temperature distribution have a serious effect on the fire resistance of restrained Q690 steel beams. A simplified calculation method, including load and critical temperature $T_{cr}$ of the restrained Q690 steel beams under fire loads and considering the several factors along with creep, is derived through regression analysis. For restrained-steel-beams, the fire resistance equation should satisfy the following:

$$M_q \leq M_{pT},$$

where the moment $M_q$ is equal to $qL^2/8$, and the restrained moment $M_{pT}$ of the restrained high-strength Q690 steel beams can be expressed as follows:

$$M_{pT} = LRf_yW_p,$$

$$LR = \frac{\beta_r\beta_{T1}a}{1 + bk\beta_{T2}(T_{cr} + T_0)^c}$$

$$k = 0.00765\left(\frac{l}{h}\right)^2 - 0.224\left(\frac{l}{h}\right) + 2.42,$$

where $W_p$ is plastic section modulus; $f_y$ is yield strength; $T_0$ and $\beta_r$ are coefficients relevant to the ratio of rotational restraint stiffness as indicated in Table 1; $\beta_{T1}$ and $\beta_{T2}$ are coefficients relevant to the cross-section temperature distribution pattern, which are 1.05 and 1.65 for the non-uniform temperature distribution pattern, respectively; both coefficients take 1.0 for the uniform temperature distribution pattern; $l/h$ is the span-to-depth ratio of the beam; $a$, $b$, and $c$ are coefficients related to $d_i/\lambda$, as indicated in Table 2.
Table 1: Values of $\beta_r$ and $T_0$.

<table>
<thead>
<tr>
<th>$\gamma$</th>
<th>0</th>
<th>0.2</th>
<th>0.6</th>
<th>1</th>
<th>&gt;5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta_r$</td>
<td>0.43</td>
<td>0.46</td>
<td>0.95</td>
<td>1</td>
<td>1.12</td>
</tr>
<tr>
<td>$T_0$</td>
<td>58</td>
<td>19</td>
<td>13</td>
<td>0</td>
<td>-3</td>
</tr>
</tbody>
</table>

Table 2: Values of $a$, $b$ and $c$.

<table>
<thead>
<tr>
<th>$di/\lambda_i$ (m$^2$/°C/W)</th>
<th>0</th>
<th>0.05</th>
<th>0.1</th>
<th>0.2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a$</td>
<td>6</td>
<td>5.95</td>
<td>5.09</td>
<td>5</td>
</tr>
<tr>
<td>$b$</td>
<td>2×10$^{-28}$</td>
<td>1.85×10$^{-28}$</td>
<td>5.47×10$^{-28}$</td>
<td>1.9×10$^{-26}$</td>
</tr>
<tr>
<td>$c$</td>
<td>10.2517</td>
<td>10.29</td>
<td>10.1</td>
<td>9.5524</td>
</tr>
</tbody>
</table>

The applicable conditions of this calculation method are as follows:
- ISO-834 standard fire is the fire curve;
- Critical temperature $T_{cr}$ is in the range between 450 °C–750 °C;
- Lateral buckling is neglected in the beams.
- The value of $di/\lambda_i$ is less than 0.2 (m$^2$/°C/W).

When the load is determined, the simplified calculation method equation for critical temperature $T_{cr}$ can be expressed as follows:

$$T_{cr} = \left[\frac{1}{bk\beta_r\beta_l} \left(\frac{\beta_r T_1 a}{LR} - 1\right) - 1\right]^{\frac{1}{c}} - T_0.$$  \hspace{1cm} (8)

Many numerical analyses were performed by Abaqus on restrained high-strength Q690 steel beams with various span-to-depth ratios, rotational restraint stiffness, and heating rates. Fig. 4 shows that numerical analyses have a good agreement with the proposed calculation design method.

![Fig. 4 Critical temperature $T_{cr}$ comparison between the design method and Abaqus.](image)

5 CONCLUSION

1. Creep has a noticeable effect on the fire resistance of restrained Q690 steel beams. Thus, ignoring creep behavior may lead to unsafe designs.

2. The presented calculation design method in this paper is suitable for calculating the critical temperature $T_{cr}$ of restrained Q690 steel beams.
ACKNOWLEDGMENTS

The authors wish to acknowledge the supports of the Natural science of foundation of Chongqing (Grant No.: cstc2018jcyjAX0596), the Fundamental Research Funds for the Central Universities (Grant No.: 2019CDQYTM027). Any opinions, findings, and conclusions or recommendations expressed in this paper are those of the authors and do not necessarily reflect the views of the sponsors.

REFERENCES

EXPERIMENTAL INVESTIGATION ON MECHANICAL PROPERTIES OF GRADE 1670 STEEL WIRES AT AND AFTER ELEVATED TEMPERATURE

Er-feng Du 1*, Xiao-bo Hu 2, Zhong Zhou 2, Jin-yu Lu 1 and Yi-qun Tang 2

1 Key Laboratory of Concrete and Prestressed Concrete Structures of Ministry of Education, National Prestress Engineering Research Center, Southeast University, Nanjing, China
E-mails: erfengdu@seu.edu.cn, 2016350071@qq.com
2 School of Civil Engineering, Southeast University, Nanjing, China
714023684@qq.com, davidjingyu@seu.edu.cn, yi-qun.tang@seu.edu.cn

Abstract: Grade 1670 steel wires were selected for elevated-temperature and post-elevated-temperature tensile tests. The test data were analyzed through comparison with the results in existing literatures. The elevated-temperature test results indicate that, mechanical properties of the steel wires degraded with the increase of temperature. The mechanical behaviors of the steel wires degraded rapidly at the temperature exceeding 300°C; and the load-carrying ability was substantially lost when the temperature increased up to 700°C. In the post-elevated-temperature test, the modulus of the steel wire was substantially completely recovered after cooling from the elevated temperatures; the nominal yield strength and ultimate strength degraded obviously after cooling from the temperature exceeding 400°C. Based on the test data, the reduction factors of the mechanical properties at and after elevated temperatures were fitted as a function of the temperature, and constitutive models of the steel wires were established. The results can provide technical supports for the analysis of fire performance of prestressed cable support structures, and their post-fire repair.

Keywords: Prestressed steel wires; Elevated temperatures; Mechanical properties; Experimental research; Stress-strain curve

DOI: 10.18057/ICASS2020.P.209

1 INTRODUCTION

Steel cables are the key members of long-span prestressed steel structures such as cable domes, string domes and tension beams. However, mechanical properties of the steel cables are significantly degraded at elevated temperatures, thereby seriously influencing the safety of the overall structure. The steel cable is mainly composed of steel wires, and thus the study on the elevated-temperature and post-elevated-temperature mechanical properties, such as the elastic modulus, proportional limit, yield strength, and ultimate strength of the steel wire, is of great significance to the fire resistance analysis and post-fire evaluation of large-span prestressed steel structures.

Mechanical properties of steel cables at elevated temperatures have been investigated by scholars and experts. Zhou et al.[1] tested mechanical behaviors of 27 Grade 1860 prestressed steel cables from ambient temperatures to 700°C, and obtained stress-strain curves of the steel cables at different temperatures. Zong et al.[2,3] conducted experimental research on mechanical performances of Grade 1860 prestressed steel cables at elevated temperatures, and obtained the degradation law and mechanical model of the steel cables. Fontanari et al.[4]
studied mechanical properties of C80 high-carbon steel cables at elevated temperatures, and provided a numerical analysis method to study the fire resistance of steel cables. Conor et al.[5] performed tensile tests on ASTM A416-12a steel cables composed of 7 steel wires at elevated temperatures, and as compared with Eurocodes, the results showed that the Eurocodes cannot accurately predict the stress-strain relationship of the steel cable. Shakya et al.[6] studied the mechanical performance of steel cables composed of 7 Grade 1860 steel wires at elevated temperatures, and gave an empirical equation for the mechanical properties of steel cables as a function of the temperature. Du et al.[7-10] carried out tensile tests at elevated temperatures on a Grade 1670 steel cable, a Grade 1670 parallel steel wire cable, and a Grade 1860 steel cable, and revealed the relationship between the reduction factor of mechanical behaviors and the temperature. Sun et al. [11-13] conducted tensile tests at elevated temperatures on a Grade 1500 stainless steel cable, a Grade 1670 high-vanadium cable, and a Grade 1770 steel wire coated with 5% zinc-aluminum alloy, and obtained a reduction factor equation for mechanical properties of steel cables and steel wires.

In view of the mechanical properties of steel cables after exposure to elevated temperatures, Fan et al.[14] studied the mechanical properties of Grade 1570 high-strength prestressed steel wires after cooling to ambient temperature from elevated temperatures, and obtained variation laws of mechanical behaviors of steel wires. Lu J. et al.[15,16] tested the post-fire mechanical performances of a Grade 1670 steel cable and prestressed steel wires of different grades, and investigated effects of different cooling methods on the mechanical behaviors of steel cables and steel wires. Zheng [17] and Atienza et al.[18] performed tensile tests on Grade 1770-grade prestressed steel wires after cooling from elevated temperatures, and obtained the degradation law of mechanical properties of steel wires after high temperature. Zong et al.[19] conducted experimental research on a Grade 1860 prestressed steel cable after heating, and revealed the relationship between the mechanical properties of the steel cable and the exposure temperature. Zhang et al.[20] carried out experimental research on the post-fire mechanical properties of a Grade 1670 cold-drawn steel wire used in suspension bridges, and analyzed the influence of the exposure temperature on the mechanical behaviors of the steel wire after cooling.

Though aforementioned scholars have performed a series of studies on the mechanical performance of steel wires, accumulated a lot of test data, due to the continuous improvement in performance of steel wires, mechanical behaviors of the newly emerging steel cables in engineering are less investigated. Therefore, this paper conducts an experimental study on the degradation law of elevated-temperature and post-elevated-temperature mechanical properties of Grade 1670 prestressed steel wires, and provides a reference for the property evaluation of prestressed steel structures under and after fire.

2 TEST PROGRAM

2.1 Test equipment

The test equipment for elevated-temperature and post-elevated-temperature mechanical properties of steel wires is shown in Figure 1. The loading device was an electronic universal testing machine, with a maximum tensile force of 300kN. The specimen was heated by an electronic elevated-temperature furnace. The furnace was equipped with multi-layer resistance wires. The furnace had a diameter of 24cm and a height of 46cm. The maximum temperature in the furnace could reach 1200°C. The strain of the steel wire was measured by an elevated-temperature strain extensometer with a gauge length of 50mm. During the
measurement, the knife edge at the end of the extensometer was closely contacting the specimen. Data such as the stress and strain were automatically collected by a computer.

![Figure 1: Electronic universal testing machine](image1)

![Figure 2: Clamping device for specimen](image2)

2.2 Test specimens

The steel wires used in the elevated-temperature and post-elevated-temperature tests had a strength grade of 1670MPa, a diameter of 7mm, and a length of 80cm. Due to the high strength and hardness of the prestressed steel wire, it is easy for the wire to slip when directly clamped by the testing machine. Therefore, pier heads and clip anchors were set at both ends of the specimen for effective clamping. The clamping device is shown in Figure 2. The diameter of the pier head was 14mm, and the diameter and length of the clip anchor were 25mm and 80mm, respectively. A length of 15cm at the upper and lower ends of the steel wire outside the elevated-temperature furnace was set aside to cool in the air, so as to avoid the influence of high temperatures on the clamping heads of the testing machine.

2.3 Loading procedure

8 temperature levels were set for the tests of elevated-temperature mechanical properties, including 20°C (ambient temperature), 100°C, 200°C, 300°C, 400°C, 500°C, 600°C, and 700°C, respectively. According to the test methods specified in “Metallic materials - Tensile testing - Part 1: Method of test at room temperature” (GB/T228.1-2010)[21] and “Metallic materials - Tensile testing - Part 2: Method of test at elevated temperature” (GB/T228.2-2015)[22], the specimens were heated to a specified temperature and then kept for 30 min, to allow the temperature to distribute uniformly in the specimens. During the heating and isothermal processes, no tension was applied on the testing machine. Stretching was performed after the temperature distributed uniformly and stably, at a constant rate of 3 mm/min.

5 temperature levels were set for the tests of post-elevated-temperature mechanical properties, including 300°C, 400°C, 500°C, 600°C, and 700°C, respectively. According to the same heating method as that in the elevated-temperature test, the specimens were firstly heated to the target temperature, subsequently kept at that temperature, and then naturally cooled to ambient temperature followed by stretching. The stretching method was the same as that in the elevated-temperature test.
3 RESULTS AND ANALYSIS OF THE ELEVATED-TEMPERATURE TEST

3.1 Visual observations

Apparent characteristics of the specimens after the elevated-temperature tensile test are shown in Figure 3. The heating temperatures of various specimens from left to right were 20°C, 100°C, 200°C, 300°C, 400°C, 500°C, 600°C and 700°C, respectively. At 20°C (ambient temperature), the specimen broke at the pier head, due to the damage near the pier head caused during cold rolling. While at other temperatures, the specimens broke in the heated area. At 20°C, the specimen was damaged at an oblique angle of 45°; at 200°C-400°C, the metallic luster of the specimen became darker, and the fracture was cup-shaped with serrations; at 500°C, the metallic luster on the surface of the specimen completely faded, and the fracture began to be tapered; at 600°C, the specimen was light yellow, and the fracture was more tapered; and at 700°C, the specimen was yellowish brown.

![Figure 3: Failure modes of steel wires after elevated-temperature test](image)

3.2 Analysis of test results

According to the stress-strain data obtained in the test, stress-strain curves of the specimens at different temperatures are plotted in Figure 4. At the same time, reduction factors of mechanical properties of the steel wires under various temperatures can be calculated, as shown in Table 1. It can be seen from the figure and the table that, with the increase of temperature, the mechanical properties such as the elastic modulus, nominal yield strength (i.e., the yield strength corresponding to the strain level of 0.2%), and ultimate strength of the specimens all degraded. The elastic modulus of the specimen remained unchanged at 100°C, and decreased at 200°C to 89% of that at ambient temperature. Above 300°C, the rate of decrease of the elastic modulus was accelerated with the temperature. At 500°C, the elastic modulus was 37% of that at ambient temperature, and at 700°C, the elastic modulus was only 2% of that at ambient temperature. As the temperature increased, the nominal yield strength and ultimate strength of the specimens decreased slowly below 200°C,
within a range of 10%, and decreased rapidly above 400°C. At 500°C, the nominal yield strength and the ultimate strength decreased by 74% and 76%, respectively. At 700°C, the nominal yield strength and ultimate strength were less than 10% of those at ambient temperature.

Table 1: Mechanical properties of prestressed steel wire at elevated temperatures

<table>
<thead>
<tr>
<th>T (°C)</th>
<th>$E_s(T)$ (GPa)</th>
<th>$E_s(T)/E_s$</th>
<th>$\sigma_{0.2}(T)$ (MPa)</th>
<th>$\sigma_{0.2}(T)/\sigma_{0.2}$</th>
<th>$\sigma_b(T)$ (MPa)</th>
<th>$\sigma_b(T)/\sigma_b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>180</td>
<td>1.00</td>
<td>1500</td>
<td>1.00</td>
<td>1703</td>
<td>1.00</td>
</tr>
<tr>
<td>100</td>
<td>180</td>
<td>1.00</td>
<td>1449</td>
<td>0.97</td>
<td>1693</td>
<td>0.99</td>
</tr>
<tr>
<td>200</td>
<td>160</td>
<td>0.89</td>
<td>1295</td>
<td>0.86</td>
<td>1656</td>
<td>0.97</td>
</tr>
<tr>
<td>300</td>
<td>148</td>
<td>0.82</td>
<td>1180</td>
<td>0.79</td>
<td>1434</td>
<td>0.84</td>
</tr>
<tr>
<td>400</td>
<td>115</td>
<td>0.64</td>
<td>1010</td>
<td>0.67</td>
<td>1108</td>
<td>0.65</td>
</tr>
<tr>
<td>500</td>
<td>66</td>
<td>0.37</td>
<td>383</td>
<td>0.26</td>
<td>414</td>
<td>0.24</td>
</tr>
<tr>
<td>600</td>
<td>31</td>
<td>0.17</td>
<td>123</td>
<td>0.08</td>
<td>155</td>
<td>0.09</td>
</tr>
<tr>
<td>700</td>
<td>3.5</td>
<td>0.02</td>
<td>40</td>
<td>0.04</td>
<td>47</td>
<td>0.03</td>
</tr>
</tbody>
</table>

Notes: $T$ is the temperature; $E_s(T)$ and $E_s$ are the elastic modulus of steel wire at temperature $T$ and ambient temperature, respectively; $\sigma_b(T)$ and $\sigma_b$ are the ultimate strength of steel wire at temperature $T$ and ambient temperature, respectively; $\sigma_{0.2}(T)$ and $\sigma_{0.2}$ are the yield strength of steel wire at temperature $T$ and ambient temperature, respectively.

In general, the mechanical performances of the prestressed steel wires changed little with the temperature below 300°C and degraded significantly at 300°C-600°C, and the load-carrying ability was substantially lost when the temperature reached 700°C.

3.3 Comparison and discussion

Reduction factors of the elastic modulus, nominal yield strength and ultimate strength obtained by test results of this paper are compared with those of a Grade 1860 prestressed steel wire of Shakya et al. [6], a Grade 1770 prestressed steel wire of Zheng et al. [16], a Grade 1860 steel cable of Du et al. [7], and a Grade 1670 high-vanadium cable of Sun et al. [12], as shown in Figures 5-7. In the figures, the reduction factors in this paper, Shakya et al. [6], Zheng et al. [16], Du et al. [7] and Sun et al. [12] are marked with DW1670, SW1860, ZW1770, DC1860 and SC1670, respectively.

(1) Comparison of reduction factors of elastic modulus

As shown in Figure 5, in terms of the steel wires, the reduction factors obtained by test results of this paper (DW1670) and ZW1770 are generally lower than SW1860; and except for 700°C, the test results of this paper are very close to ZW1770. As compared with the test results of steel cables, DW1670 is close to DC1860 at 100°C, 500°C and 600°C, and lower than DC1860 at other temperatures; close to SC1670 below 400°C, and lower than SC1670 above 500°C.
Figure 5: Comparison of reduction factors of elastic modulus

(2) Comparison of reduction factors of nominal yield strength
As can be seen in Figure 6, DW1670 and SW1860 are generally higher than ZW1770; DW1670 is close to SW1860 below 200°C, higher than SW1860 at 300°C and 400°C, and lower than SW1860 above 500°C; close to DC1860 except for 400°C at which DW1670 is higher than DC1860; and lower than SC1670 except for 100°C at which DW1670 is close to SC1670.

(3) Comparison of reduction factors of ultimate strength
As exhibited in Figure 7, the reduction factors obtained by test results of this paper (DW1670) are higher than SW1860 and ZW1770 from 200°C to 400°C, and close to ZW1770 but lower than SW1860 above 500°C; higher than SC1670 and DC1860 at 300°C and 400°C, and close to SC1670 and DC1860 from 500°C to 700°C except for 500°C at which DW1670 is lower than SC1670.

In general, though the reduction factors of mechanical properties of prestressed steel wires and cables of different strength grades are discrete to a certain extent, their variation laws are basically the same. The mechanical properties of steel cables under elevated temperature can be approximately represented by those of steel wires.

Figure 6: Comparison of reduction factors of nominal yield strength

Figure 7: Comparison of reduction factors of ultimate strength

Figure 8: Reduction factor of elastic modulus - temperature curve

3.4 Fitting equation for reduction factors
According to the test data, fitting equations for reduction factors of mechanical properties of steel wires at high temperatures are obtained as follows:
3.4.1 Elastic modulus

\[ \frac{E_s(T)}{E_s} = \frac{1}{1 + e^{0.0101(T-550)}} \]  

(1)

where, \( E_s(T) \) and \( E_s \) are the elastic modulus of steel wires at temperature \( T \) and ambient temperature, respectively.

Figure 8 gives the fitting curve of Eq. (1).

3.4.2 Proportional limit

\[ \frac{\sigma_p(T)}{\sigma_p} = 7.46 \times 10^{-9} T^3 - 8.94 \times 10^{-6} T^2 + 1.23 \times 10^{-3} T + 0.97 \]  

(2)

where, \( \sigma_p(T) \) and \( \sigma_p \) are the proportional limit of steel wires at temperature \( T \) and ambient temperature, respectively.

The fitting curve of Eq. (2) is plotted in Figure 9.

3.4.3 Ultimate strength

\[ \frac{\sigma_b(T)}{\sigma_b} = \frac{1}{1 + e^{0.0147(T-432)}} \]  

(3)

where, \( \sigma_b(T) \) and \( \sigma_b \) are the ultimate strength of steel wires at temperature \( T \) and ambient temperature, respectively.

Figure 10 shows the fitting curve of Eq. (3).

3.4.4 Strain at ultimate strength

The strain corresponding to the maximum stress of the specimen reaching the ultimate strength is the strain at ultimate strength. The reduction factor of the strain at ultimate strength at elevated temperature is obtained as follows:

\[ \frac{\varepsilon_b(T)}{\varepsilon_b} = \begin{cases} 1 & 20^\circ C \leq T \leq 180^\circ C \\ -3.30 \times 10^{-3} T + 1.57 & 180^\circ C \leq T \leq 400^\circ C \\ 3.40 \times 10^{-4} T + 9.67 \times 10^{-2} & 400^\circ C \leq T \leq 500^\circ C \\ 5.80 \times 10^{-3} T + 2.62 & 400^\circ C \leq T \leq 700^\circ C \end{cases} \]  

(4)

where, \( \varepsilon_b(T) \) and \( \varepsilon_b \) are the strain at ultimate strength of steel wires at temperature \( T \) and ambient temperature, respectively.
ambient temperature, respectively.

The fitting curve of Eq. (4) is shown in Figure 11.

![Figure 11: Reduction factor of strain at ultimate strength - temperature curve](image1)

![Figure 12: Reduction factor of nominal yield strength - temperature curve](image2)

3.4.5 Nominal yield strength

\[
\frac{\sigma_{0.2}(T)}{\sigma_{0.2}} = \frac{1}{1 + e^{0.0135(T-424)}}
\]

(5)

where, \(\sigma_{0.2}(T)\) and \(\sigma_{0.2}\) are nominal yield strength of steel wires at temperature \(T\) and ambient temperature, respectively.

The fitting curve of Eq. (5) is shown in Figure 12.

3.4.6 Strain at nominal yield strength

\[
\frac{\varepsilon_{0.2}(T)}{\varepsilon_{0.2}} = \begin{cases} 
1 & 20^\circ C \leq T \leq 380^\circ C \\
2.40 \times 10^{-3}T + 1.9161 & 380^\circ C < T \leq 600^\circ C
\end{cases}
\]

(6)

where, \(\varepsilon_{0.2}(T)\) and \(\varepsilon_{0.2}\) are strain at nominal yield strength of steel wires at temperature \(T\) and ambient temperature, respectively.

The fitting curve of Eq. (6) is presented in Figure 13.

![Figure 13: Reduction factor of strain at nominal yield strength - temperature curve](image3)

![Figure 14: Stress-strain fitting curves of steel wires at elevated temperatures](image4)

3.5 Constitutive equation

The constitutive equation of the steel wires at elevated temperatures is fitted with a trilinear line. The trilinear line includes a segment from the origin to the proportional limit point, a segment from the proportional limit point to the nominal yield strength point, and a segment
from the nominal yield strength point to the ultimate strength point. The fitting equation is shown as follows:

\[
\sigma = \begin{cases} 
E_s(T) \times \varepsilon & 0 \leq \varepsilon \leq \frac{\sigma_p(T)}{E_s(T)} \\
\frac{\sigma_{0.2}(T) - \sigma_p(T)}{\varepsilon_{0.2}(T) - \frac{\sigma_p(T)}{E_s(T)}} \times \left[ \varepsilon - \frac{\sigma_p(T)}{E_s(T)} \right] + \sigma_p(T) & \frac{\sigma_p(T)}{E_s(T)} < \varepsilon < \varepsilon_{0.2}(T) \\
\frac{\sigma_b(T) - \sigma_{0.2}(T)}{\varepsilon_b(T) - \varepsilon_{0.2}(T)} \times \left[ \varepsilon - \varepsilon_{0.2}(T) \right] & \varepsilon_{0.2}(T) < \varepsilon < \varepsilon_b(T) 
\end{cases}
\]  

(7)

Stress-strain curves at various temperatures obtained by the equation are shown in Figure 14.

4 RESULTS AND ANALYSIS OF POST-ELEVATED-TEMPERATURE TEST

4.1 Visual observations

Figure 15 shows the failure modes of the specimens after the tensile test are plotted in. The maximum temperatures experienced by various specimens from left to right were 300°C, 400°C, 500°C, 600°C and 700°C, respectively. When the specimen was experiencing a temperature of 300°C, it was broken near the pier head, with a fracture along the 45° direction, which phenomenon was the same as the specimen broken at ambient temperature. The specimens that had been exposed to a temperature above 400°C were all broken in the heated area after cooling, and their fractures were all tapered and necked. After exposure to 500°C, the metallic luster on the surface of the specimen faded. The surface of the specimen turned light yellow after exposure to 600°C, and yellowish - brown after exposure to 700°C.

![Figure 15: Failure modes of steel wires after post-elevated-temperature test](image)

4.2 Analysis of test results

Stress-strain curves of the specimens cooled from different temperatures are plotted in Figure 16. According to the test data, reduction factors of mechanical properties of the steel wires cooled from different temperatures can be obtained, as tabulated in Table 2. It can be found from the figure and table that, there is a yield plateau in the stress-strain curve of the specimen at ambient temperature and after cooling from higher temperatures, and the stress
decreases with the increase of temperature when the yield plateau is reached. The elastic modulus of the specimens does not change obviously after cooling from the elevated temperatures, and their reduction factors are all within 10%. Therefore, the specimens can recover the initial elastic modulus after fire. After cooling from the temperature below 400°C, the nominal yield strength and ultimate strength of the specimens had no obvious degradation, and the reduction factor was within 10%. The nominal yield strength and ultimate strength of the specimens began to degrade significantly after cooling from the temperature above 400°C. The nominal yield strength decreased by 22% and the ultimate strength decreased by 23% after cooling from 500°C. With the increase of the maximum exposure temperature, the degradation of the nominal yield strength and ultimate strength of the specimen was more obvious. After cooling from the temperature up to 700°C, the nominal yield strength and the ultimate strength had decreased to 32% and 40% of that at the ambient temperature, respectively.

Table 2: Mechanical properties of prestressed steel wire after cooling from elevated temperatures

<table>
<thead>
<tr>
<th>$T$ (°C)</th>
<th>$E^*_S(T)$ (GPa)</th>
<th>$E^*_S(T)/E_S$</th>
<th>$\sigma^\prime_{0.2}(T)$ (MPa)</th>
<th>$\sigma^\prime_{0.2}(T)/\sigma_{0.2}$</th>
<th>$\sigma^\prime_b(T)$ (MPa)</th>
<th>$\sigma^\prime_b(T)/\sigma_b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>180</td>
<td>1.00</td>
<td>1500</td>
<td>1.00</td>
<td>1703</td>
<td>1.00</td>
</tr>
<tr>
<td>300</td>
<td>175</td>
<td>0.97</td>
<td>1490</td>
<td>0.99</td>
<td>1705</td>
<td>1.00</td>
</tr>
<tr>
<td>400</td>
<td>185</td>
<td>1.03</td>
<td>1503</td>
<td>1.00</td>
<td>1731</td>
<td>1.02</td>
</tr>
<tr>
<td>500</td>
<td>173</td>
<td>0.96</td>
<td>1175</td>
<td>0.78</td>
<td>1306</td>
<td>0.77</td>
</tr>
<tr>
<td>600</td>
<td>171</td>
<td>0.95</td>
<td>929</td>
<td>0.62</td>
<td>961</td>
<td>0.56</td>
</tr>
<tr>
<td>700</td>
<td>190</td>
<td>1.06</td>
<td>485</td>
<td>0.32</td>
<td>683</td>
<td>0.40</td>
</tr>
</tbody>
</table>

Notes: $T$ is the temperature, $E^*_S(T)$ is the elastic modulus of steel wire restored to ambient temperature from $T$, $\sigma^\prime_{0.2}(T)$ is the yield strength point strain of steel wire restored to ambient temperature from $T$, $\sigma^\prime_b(T)$ is the ultimate strength of steel wire restored to ambient temperature from $T$.

4.3 Comparison and discussion

Since the elastic modulus of the steel wires did not change significantly after cooling from elevated temperatures, only the reduction factors of the nominal yield strength and ultimate strength of the steel wires are compared. Reduction factors of the nominal yield strength and ultimate strength obtained in this paper are compared with those of a Grade 1570 prestressed steel wire of Fan et al. [14], a Grade 1670 galvanized prestressed steel wire of Lu et al. [15], and a Grade 1770 prestressed steel wire of Zheng et al. [17] and Atienza et al. [18], as exhibited in Figures 17 and 18. In the figures, the reduction factors in this paper, Fan et al. [14], Lu et al. [15], Zheng et al. [17] and Atienza et al. [18] are marked with DW1670, FW1570, LW1670, ZW1770 and AW1770, respectively.
(1) Comparison of nominal yield strength

As can be seen in Figure 17, the reduction factors of this paper (DW1670) are close to those of LW1670, ZW1770 and AW1770 after cooling from temperatures below 300°C; close to LW1670 and higher than others after cooling from 400°C; close to LW1670 and AW1770 and higher than ZW1770 and FW1570 after cooling from 500°C; and only lower than LW1670 after cooling from 600°C. DW1670 decreases more quickly after cooling from 600°C-700°C, and is obviously lower than LW1670 and close to the others after cooling from 700°C.

(2) Comparison of ultimate strength

As shown in Figure 18, the reduction factors of FW1570 are obviously low. DW1670 is close to the others after cooling from temperatures below 300°C; close to LW1670 and higher than the others after cooling from 400°C; slightly lower than LW1670, close to AW1770, and slightly higher than ZW1770 after cooling from 500°C; and close to ZW1770 and AW1770 and lower than LW1670 after cooling from temperatures above 600°C.

In general, except for FW1570, the nominal yield strength and ultimate strength of different grades of steel wires begin to degrade significantly when the exposure temperature reaches above 400°C, and the decay trends are essentially consistent.

![Figure 17: Comparison of reduction factors of nominal yield strength](image)

![Figure 18: Comparison of reduction factors of ultimate strength](image)

### 4.4 Fitting equations for reduction factors

According to the test data, fitting equations for reduction factors of mechanical properties of steel wires after exposed to elevated temperature are given as follows:

#### 4.4.1 Elastic modulus

\[
\frac{E_s^r(T)}{E_s} = 1
\]  

where, \( E_s^r(T) \) and \( E_s \) are the elastic modulus of the steel wire restored from temperature \( T \) to ambient temperature, and the initial elastic modulus of the steel wire, respectively.

#### 4.4.2 Proportional limit

\[
\frac{\sigma_p^r(T)}{\sigma_p} = \begin{cases} 
1 & 20^\circ C \leq T \leq 400^\circ C \\
-0.0019T + 1.7591 & 400^\circ C < T \leq 700^\circ C 
\end{cases}
\]  

where, \( \sigma_p^r(T) \) and \( \sigma_p \) are the proportional limit of the steel wire restored from temperature \( T \)
to ambient temperature, and the initial proportional limit of the steel wire, respectively.

The fitting curve of Eq. (9) is plotted in Figure 19.

![Figure 19: Reduction factor of proportional limit - temperature curve](image1)

![Figure 20: Reduction factor of ultimate strength - temperature curve](image2)

4.4.3 Ultimate strength

\[
\frac{\sigma_{b}^\#(T)}{\sigma_{b}} = \begin{cases} 
1 & \text{20°C} \leq T \leq 400^\circ \text{C} \\
-0.002T + 1.814 & \text{400^\circ \text{C} < T \leq 700^\circ \text{C}}
\end{cases}
\]  

(10)

where, \(\sigma_{b}\) and \(\sigma_{b}^\#\) are the ultimate strength of the steel wire restored from temperature \(T\) to ambient temperature, and the initial ultimate strength of the steel wire, respectively.

The fitting curve of Eq. (10) is shown in Figure 20.

4.4.4 Nominal yield strength

\[
\frac{\sigma_{0.2}^\#(T)}{\sigma_{0.2}} = \begin{cases} 
1 & \text{20°C} \leq T \leq 400^\circ \text{C} \\
-0.0022T + 1.892 & \text{400^\circ \text{C} < T \leq 700^\circ \text{C}}
\end{cases}
\]  

(11)

where, \(\sigma_{0.2}\) and \(\sigma_{0.2}^\#\) are the nominal yield strength of the steel wire restored from temperature \(T\) to ambient temperature, and the initial nominal yield strength of the steel wire, respectively.

Figure 21 presents the fitting curve of Eq. (11).

![Figure 21: Reduction factor of nominal yield strength - temperature curve](image3)

![Figure 22: Reduction factor of strain at nominal yield strength - temperature curve](image4)

4.4.5 Strain at nominal yield strength
\[
\varepsilon_{0.2}^{*}(T) = \begin{cases} 
1 & 20^\circ C \leq T \leq 400^\circ C \\
-0.0018T + 1.7591 & 400^\circ C < T \leq 700^\circ C 
\end{cases}
\]  
(12)

where, \( \varepsilon_{0.2} \) and \( \varepsilon_{0.2}^{*} \) are the strain at nominal yield strength of the steel wire restored from temperature \( T \) to ambient temperature, and the initial strain at nominal yield strength of the steel wire, respectively.

The fitting curve of Eq. (12) is plotted in Figure 22.

4.5 Constitutive equation

By the same method as that at elevated temperatures, the constitutive equation of the steel wire after cooling from high temperatures is fitted with a trilinear line. The fitting equation is shown as follows:

\[
\sigma = \begin{cases} 
E_s^*(T) \times \varepsilon & 0 \leq \varepsilon \leq \frac{\sigma_p^*(T)}{E_s^*(T)} \\
\frac{\sigma_{0.2}^*(T) - \sigma_p^*(T)}{\varepsilon_{0.2}^*(T) - E_s^*(T)} \times \left[ \varepsilon - \frac{\sigma_p^*(T)}{E_s^*(T)} \right] + \sigma_p^*(T) & \frac{\sigma_p^*(T)}{E_s^*(T)} \leq \varepsilon \leq \varepsilon_{0.2}^*(T) \\
\frac{\sigma^*(T) - \varepsilon_{0.2}^*(T)}{\varepsilon_{0.2}^*(T) - E_s^*(T)} \times \left[ \varepsilon - \varepsilon_{0.2}^*(T) \right] + \sigma_{0.2}^*(T) & \varepsilon_{0.2}^*(T) \leq \varepsilon \leq \varepsilon_b^*(T) 
\end{cases}
\]  
(13)

Stress-strain curves of steel wires cooling from various temperatures obtained by the equation are exhibited in Figure 23.

![Stress-strain fitting curves of steel wire after cooling from elevated temperatures](image)

Figure 23: Stress-strain fitting curves of steel wire after cooling from elevated temperatures

5 CONCLUSIONS

In this paper, elevated-temperature and post-elevated-temperature tensile tests were performed on Grade 1670 steel wires with a diameter of 7mm. The test results were compared with the existing test results of prestressed steel wires and steel cables. The following conclusions are obtained:

(1) At elevated temperatures: mechanical properties of the prestressed steel wire changed little with the temperature below 300°C and degraded significantly at 300°C-600°C, and the load-carrying ability was substantially lost when the temperature reached 700°C.

(2) After elevated temperatures: the elastic modulus of the steel wires had no obvious
change with the exposure temperature; the nominal yield strength and ultimate strength of the steel wires remained unchanged after cooling from exposure temperatures below 400°C, and tended to decrease after cooling from exposure temperatures above 400°C.

(3) Though the reduction factors of mechanical properties of steel wires and steel cables given in the literatures are discrete to a certain extent, their variation laws are basically the same. The elevated-temperature mechanical properties of steel cables can be approximately represented by those of steel wires.

(4) Equations of elevated-temperature and post-elevated-temperature mechanical behaviors of steel wires are fitted as a function of the temperature, and constitutive models of the steel wire at elevated temperatures and after exposed to elevated temperatures are obtained.

ACKNOWLEDGMENTS

The research work described in this paper is sponsored by the Nation Natural Science Foundation of China (Grant No. 51808117). The financial support is highly appreciated.

REFERENCES


FINITE ELEMENT SIMULATION FOR ULTRA-HIGH-PERFORMANCE CONCRETE-FILLED DOUBLE-SKIN TUBES EXPOSED TO FIRE

A.H.A. Abdelrahman1,* , Mohamed Ghannam1, S. Lotfy2, and Mohammad AlHamaydeh 3

1 Structural Engineering Department, Faculty of Engineering, Mansoura University, Egypt
E-mails: a_hussain@mans.edu.eg, m.ghannam@mans.edu.eg

2 Civil Engineering Department, MISR Higher Institute for Engineering and Technology, Mansoura, Egypt
E-mail: same7.lotfy@gmail.com

3 Department of Civil Engineering, American University of Sharjah, PO Box 26666, Sharjah, United Arab Emirates. Email: malhamaydeh@aus.edu

Abstract: Ultra-high-performance concrete (UHPC) or ultra-high-strength concrete (UHSC) are alternatively used to reduce construction materials, thereby achieving more sustainable constructions. Moreover, engaging the advantages of concrete cores and outer steel tubes in concrete-filled steel tubes (CFST) or ductile concrete-filled double-skin tubes (CFDST) is of great interest for the better performance of such members under fire. Nevertheless, current design provisions do not provide design models for UHPC-filled double-skin tubes under fire, and existing finite-element (FE) methodologies available in the literature may not accurately simulate the behaviour of CFDST exposed to fire. Therefore, this paper develops a comprehensive FE protocol implementing the scripting technique to model CFDST members for heat transfer and coupled (simultaneously or sequentially) thermal-stress analyses. Various modelling parameters incorporated in the proposed FE routine include the cross-sectional geometry (circular, elliptical, hexagonal, octagonal, and rectangular), the size (width, diameter, and wall thickness), interactions, meshing, thermal- and mechanical-material properties, and boundary conditions. The detailed algorithm for heat transfer analysis is presented and elaborated via a flow chart. Validations, verifications, and robustness of the developed FE models are established based on extensive comparison studies with existing fire tests available in the literature. As a result, and to recognize the value of the current FE methodology, an extensive parametric study is conducted for different affecting parameters (e.g., nominal steel ratio, hollowness ratio, concrete cylindrical strength, yield strength of metal tubes, and width-to-thickness ratio). Extensive FE results are used for optimizing the fire design of such members. Consequently, a simplified and accurate analytical model that can provide the axial load capacity of CFDST columns under different fire ratings is presented.

Keywords: Thermal-stress analysis; Ultra-high-performance concrete; High-strength steel; Double-skin tubes; Heat transfer; Fire design.

DOI: 10.18057/ICASS2020.P.263

1. INTRODUCTION

Ultra-high-performance concrete (UHPC) or ultra-high-strength concrete (UHSC) are alternatively used to reduce construction materials, thereby achieving more sustainable constructions. Different design codes define the limits in which the high-strength concrete (HSC) can be classified. For example, ACI [1] provides a cylindrical compressive strength of 50 MPa to define the HSC, while UHPC is defined as the concrete with a specified compressive strength of at least 150 MPa and specific requirements regarding durability, ductility, and
toughness. On the other hand, EN 1992-1-2 [2] capes the maximum concrete characteristic strength to 90 MPa for the fire-resistant design. Generally, UHSC exhibits poor ductility, low seismic resistance, and explosive spalling problems under fire. Nevertheless, it shows significant improvements when filling steel tubes in concrete-filled steel tubes (CFST) or ductile concrete-filled double-skin tubes (CFDST) due to the confinement effect.

In the past few decades, ductile CFDST columns, engaging the advantages of steel-concrete-steel sandwich and concrete-filled tubes (CFT), have been extensively utilized, and hence, given a great deal of research [3-9]. Compared to research conducted on CFST, recent research is devoted to investigating the behavior of CFDST under fire [10-15]. CFDST columns behave much better than the traditional CFST under fire. The inner tube has relatively lower temperatures due to the surrounding concrete protection, usually leading to longer sustaining of the structural loads under fire. Therefore, the temperature development within the cross-section and stiffness degradation due to higher temperatures are of great concern when analyzing such members under fire. To this end, various methods can be utilized to predict temperature development, such as the finite-difference (FD) method [16], finite-element (FE) method [17-18], or a combined method [19]. Even though the FE method might be cumbersome and time-consuming, the scripting technique prioritizes this method for the heat transfer analysis of composite members. Thus, extensive configurations and combinations of members’ profiles (e.g., different sectional shapes for double tubes and different materials for inner and outer cores and skins) can be effectively and conveniently modelled. It is noted that most of the conducted research on the fire performance of concrete-filled tubes focused on circular, square, and rectangular sections of CFST and CFDST [15-17]. However, distinct behaviors regarding the confinement effect and temperature development are observed for different cross-sectional shapes. This paper emphasizes extensive configurations of CFDST members considering different cross-sectional profiles, such as those in Figure 1, and different materials for the outer tube, the inner tube, and the infilling concrete. Since the ideal engagements of different alternative materials and the cross-sectional shapes for outer and inner tubes can provide better performance under fire, they are comprehensively investigated in the current study.

![Figure 1. Concrete-filled double-tubes with different shapes in composite constructions.](image-url)

**Shapes:**
- C = Circular
- S = Square
- E = Elliptical
- H = Hexagonal
- O = Octagonal

**Filling:**
- DS = Double Skin
- DT = Double Tube

**Example:**
- CH-DS = Circular Hexagonal and Double Skin

The main objective of this paper is to validate a FE modelling protocol to simulate CFDST members for heat transfer analysis under fire. Different cross-sectional shapes are modelled, including circular, square, rectangular, elliptical, hexagonal, and octagonal. A review of the existing FE models and experimental database for concrete-filled tubes is conducted first and utilized for verification purposes. Further, scripting and automation techniques are employed for efficient and convenient FE modelling. Various parameters are incorporated, such as
material definitions, geometry (i.e., cross-section type and size), meshing control, boundary conditions, and thermal contact between metal tubes and filling concrete. As a sequel, thermal properties of carbon-steel, stainless-steel, and different infilling-concrete grades (i.e., NSC, HSC, and UHSC) are included and utilized with extensive combinations. Moreover, a refined model for the thermal conductance between metal tubes and concrete cores is provided for the octagonal and hexagonal profiles. Finally, an extensive parametric study is conducted, and design guidelines are to be presented based on the extensive database available in the current study.

2. FINITE ELEMENT MODELING OF CFDST COLUMNS

2.1 General

In simulating the structural members under fire, whereas the thermal-structural response is investigated, the heat transfer analysis is crucial for a sequentially coupled thermal stress analysis. When the stress field is only dependent on the temperature field, all structural-related aspects such as axial load, mechanical material models, and end restraints are not considered. In the current study, FE models based on commercial software ABAQUS and adopting the scripting technique are developed. In simulating CFDST members, 8-node continuum heat transfer elements (DC3D8) are adopted for infilling concrete cores (i.e., the inner core and the concrete ring, see Figure 2). Meanwhile, 4-node shell-elements (DS4) are utilized for modelling the double-skin tubes. Figure 2 shows a typical FE model for heat transfer analysis of CFDST members.

Moreover, the convective heat transfer coefficient ($\alpha_c$) and emissivity coefficient ($\epsilon_m$) are essential when simulating heat transfer by convection and radiation mechanisms, respectively [21]. Different researchers have proposed different values for ($\alpha_c$) and ($\epsilon_m$) in conducting heat-transfer analysis based on extensive parametric studies [17, 21]. Because both $\alpha_c$ and $\epsilon_m$ are dependent on the physical and thermal properties of the heated surface, complexities and uncertainties are accompanied with the calculations or assumptions. In the current study, the recommended values of $\alpha_c$ (25 or 35 $W/m^2K$ for carbon-steel and stainless-steel, respectively) are adopted. Meanwhile, the corresponding emissivity coefficients ($\epsilon_m$) of 0.7 and 0.4 are utilized for the outer surface made of carbon-steel and stainless-steel [17].

![Figure 2. Typical finite element model for heat transfer analysis of CFDST.](image)
2.2 Thermal properties of different materials

Because the thermal properties when simulating the heat-transfer analysis are of great interest, especially for different concrete grades (i.e., NSC, HSC, and UHSC), such properties are thoroughly reviewed. A great deal of research is conducted to investigate the effect of temperature on such thermal properties, and some of them are herein presented [22-27]. Consequently, the material density, thermal conductivity, and specific heat for different concrete grades, and their constitutive models are presented here. For more details, the reader is kindly referred to the paper by the authors [28].

2.2.1 Concrete materials (NSC, HSC, and UHSC)

This section describes the thermal properties models of the concrete materials used in the present FE models. Analysis for three types of concrete (i.e., conventional normal strength concrete (NSC), lightweight concrete (LWC), and high strength concrete (HSC) or ultra-high strength concrete (UHSC)) is presented. Eurocode-2 [2] provides a classification for NSC and HSC, which stated that NSC is up to cylinder compressive strength of 50 MPa; three classes were used for HSC ranging from 55 to 90 MPa. The strength for UHSC is recommended to be more than 90 MPa.

(a) Density

Concrete density decreases with increasing temperature [29]. The density of concrete ($\rho_c$) at elevated temperatures depends on its moisture content, its aggregate materials, and method of curing and temperature. Generally, siliceous concrete aggregate, such as quartz, has a steeper decrease of density with increasing temperature than the decrease of that of limestone concrete due to the thermal expansion, which is higher in siliceous concrete than in the case of limestone concrete [22].

Most analytical models assume that concrete density is not affected when temperature increases. Nevertheless, Eurocode-2 [2] provides a temperature-dependent concrete density affected by water loss from the concrete at elevated temperatures. A comparison between different models of concrete density (Eurocode-2 [2], ASCE [30], and AIJ [31]) is shown in Figure 3. As can be seen in Figure 3(a), ASCE [30] and AIJ [31] assume that concrete density has a constant value (temperature-independent). Oppositely, Eurocode-2 [2] proposes a temperature-dependent model for concrete density in which the density decreases gradually with increasing temperature, which is adopted in this study. For lightweight concrete, ASCE [30] and Eurocode-4 [2] assumed constant density with temperature.

![Figure 3. Concrete density at elevated temperature.](image)
(b) Thermal conductivity

Various parameters affecting the thermal conductivity of concrete include water content, cement paste, and pore volume and distribution. Previous research on thermal conductivity [22, 29] found that thermal conductivity generally decreases with increasing temperature. Different models that simulate concrete thermal conductivity are identified in Figure 4, wherein the ASCE model gives different values for concrete thermal conductivity depending on the aggregate type (carbonate or siliceous). Eurocode-2 [2] model shows upper and lower conductivity limits and recommends the upper limits to be adopted for CFST columns since these values were obtained from test results on composite cross-sections. The model proposed by Tao and Ghannam [17] is adopted in this study for NSC (with compressive cylinder strength less than or equal to 65 MPa) and Eurocode-4 [2] for LWC. The accuracy of three models (i.e., Kodur and Sultan, 2003 [25] for HSC, Khaliq and Kodur, 2011 [26] for HSC, and Kodur, et al., 2020 [27] for UHSC) are examined in this paper to simulate HSC or UHSC (with compressive cylinder strength of more than 65 MPa).

![Concrete thermal conductivity](image)

Figure 4. Concrete thermal conductivity; (a) NSC, (b) LWC, and (c) HSC & UHSC.

(c) Specific heat

The aggregate type, density, and moisture content have influenced the specific heat of concrete [26]. Different models that simulate concrete’s specific heat are identified in Figure 5, in which the ASCE model gives different values for concrete specific heat depending on the aggregate type. On the other hand, Eurocode-2 uses one model for all types of aggregate, wherein the effect of water content is taken into account. A peak value of the specific heat between 100°C and 115°C is provided, and a linear decrease between 115°C and 200°C is assumed. For water content of 3%, a peak value of 2020 J/kg.K is used, while for 10% water
content, a value of 5600 J/kg.K is considered. At high temperatures, the chemically bound water in the concrete is released and evaporated as free water. The ASCE model takes into account the effect of bond water released from the concrete, as can be observed in Figure 5(a), wherein the peak values between 400°C and 600°C for concrete with siliceous aggregate, and between 600°C and 800°C for concrete with carbonate aggregate are provided.

In the present study, Eurocode-4 is adopted for the specific heat of NSC (with compressive cylinder strength less than or equal to 65 MPa). Similar to the thermal conductivity, the three models proposed for HSC and UHSC, as shown in Figure 5(c), are examined for concrete with a compressive cylinder strength of more than 65 MPa. Meanwhile, ASCE [52] model is used to simulate the specific heat of LWC, see Figure 5(b).

![Figure 5](image_url)

Figure 5. Concrete specific heat; (a) NSC, (b) LWC, and (c) HSC.

### 2.3 Thermal contact conductance at the interaction surfaces

The thermal contact conductance at the interaction surfaces of metal tubes and concrete cores is crucial. Ding and Wang [32] reported that an air gap developed between steel tubes and concrete, which can be attributed to: 1) the difference in thermal expansion between steel and concrete materials, 2) the difference in temperature development throughout steel and concrete cross-sections, and 3) the presence of initial gap due to concrete shrinkage. Accordingly, thermal contact conductance \( (h_j) \) between steel and concrete surfaces can represent the heat transfer between the two materials. Ghojel [33-34] studied the thermal conductance between steel and concrete experimentally and analytically for square and circular columns. An equation that relates the heat conductance with steel temperature for square and circular columns was proposed and revised later by Ghojel [33] as presented in Eq. 1, in which \( h_j \) is the heat conductance and \( T_s \) is the steel surfaces temperature.
Tao and Ghannam [17] proposed a model that predicts the value of $h_j$ based on the cross-section type and the outer dimension of the CFST columns (Eq. 2)

$$h_j = a(D/100)^b \text{ in W/m}^2 \text{K}$$  \hspace{1cm} (2)

where $D$ is the outer dimension of the columns in (mm), and $a$ and $b$ are taken as 516 and 2.373 for circular columns, and 115 and 0.85 for square columns, respectively. Constant values of 45.2 and 38.1 (W/m$^2$ K) are, respectively, used for the heat conductance in square and circular columns having an outer diameter greater than 300 mm.

It can be noticed from the existing models [17, 33-34] that $h_j$ is dependent on the cross-section shape. This can be explained by the difference in temperature development throughout the square and circular cross-sections, where square columns are heated much faster at the corners than the mid-width of the column side, resulting in variation of the gap between steel tube and concrete core. Oppositely, the cross-section is heated evenly from all sides in circular columns. Another reason is changing the location and shape of the air gap due to local buckling of steel tubes under loading, which is higher in square columns than circular columns.

In this study, the effect of heat conductance on the temperature field of CFDST columns has been investigated numerically using the proposed FE models. The heat conductance between the outer tube and outer concrete core has been used as adopted by Tao and Ghannam [17]. A sensitivity analysis using FE models has been conducted to investigate the effect of heat conductance between the inner tube and the outer concrete core, and between the inner tube and the inner core. Based on the fire tests presented in [28], Table 1 shows the effect of different values of heat conductance on the temperature development of point 3 (inner steel tube). It can be noticed that applying Eq. 2, which was proposed by Tao and Ghannam [17], can be used to simulate the interaction between the inner tube and concrete core and between the inner tube and outer concrete, giving reasonable predictions for inner tube temperature. The mean test-to-predicted value is 1.031 with a coefficient of variance (COV) of 0.283.

<table>
<thead>
<tr>
<th>Results for $T_{test}/T_{FF}$</th>
<th>Eq. 2</th>
<th>$h_j = 50$</th>
<th>100</th>
<th>200</th>
<th>Tie</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>1.030</td>
<td>1.127</td>
<td>1.052</td>
<td>1.014</td>
<td>1.012</td>
</tr>
<tr>
<td>COV</td>
<td>0.283</td>
<td>0.367</td>
<td>0.342</td>
<td>0.331</td>
<td>0.326</td>
</tr>
</tbody>
</table>

Note: Tie means adopting the Tie constraints between the interaction surfaces.

In order to predict the temperature of the inner tube of octagonal and hexagonal CFDST columns, an interpolation for the coefficients presented in Eq. 2 is performed. The interpolation is based on the coefficients of circular and square columns. The ratio between the area of the inscribed circle of the specified shape (i.e., octagonal and hexagonal) and the actual area of this shape ($A/A_{circle}$) is taken as the interpolation criteria. Such ratio equals 1.0 for circular sections and maximum for square sections. Accordingly, the interpolation results are as follows; $a$ and $b$ are taken as 435.6 and 2.184 for octagonal columns, respectively, and 365.3 and 1.996 for hexagonal columns. Constant values of 39.5 and 40.8 (W/m$^2$ K) are used for octagonal and hexagonal columns, respectively, with a max dimension of 300 mm. As a result, the values of heat conductance for different dimensions and different cross-sections adopting Eq. 2 are shown in Figure 6. It is worthwhile mentioning that the new coefficients proposed for octagonal and hexagonal columns are based on analytical studies only due to the dearth of available test results.
for octagonal and hexagonal columns. Hence, more experimental studies need to be conducted in order to verify these values.

![Figure 6. Values of heat conductance for different dimensions and different cross-sections.](image)

Finally, details of the proposed FE algorithm, including the generation of the input data file, conducting the heat transfer analysis, and post-processing of the analysis results, are illustrated in detail with a flowchart as shown in Figure 7.

![Figure 7. Algorithm for automated Heat Transfer analysis for CFST and CFDST members](image)

3. VALIDATION AND VERIFICATION OF THE PROPOSED ROUTINE

The proposed FE model has been verified against available fire tests on concrete-filled double tube/skin columns [28]. Owing to space limitations, only selected comparisons with the available fire tests are presented in this section. Table 2 shows the result obtained for the simulation of NSC, where the mean test-to-predicted temperatures and the corresponding coefficient of variance (COV) are provided for the 4 points within the cross-sections (i.e., point
1 is on the outer steel tube, point 2 is at the middle of the concrete ring, point 3 is for the inner tube, and point 4 is at the center of concrete core).

### Table 2. Results of FE model verification for NSC

<table>
<thead>
<tr>
<th></th>
<th>Point 1</th>
<th>Point 2</th>
<th>Point 3</th>
<th>Point 4</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Mean</strong></td>
<td>1.102</td>
<td>1.166</td>
<td>1.070</td>
<td>0.851</td>
</tr>
<tr>
<td><strong>COV</strong></td>
<td>0.12</td>
<td>0.163</td>
<td>0.375</td>
<td>0.080</td>
</tr>
</tbody>
</table>

Moreover, Table 3 compares the three different thermal models used to simulate the heat transfer in HSC and UHSC at the four points within the cross-sections. Observing point 4 located at the center of the inner concrete core, the mean test-to-FE results, when adopting the recently proposed model by Kodur et al. (2020) [27], is 0.822 with a COV of 0.313, while the corresponding values are 0.672 and 0.404, respectively, when the models by Kodur and Sultan (2003) [25] are utilized. From these comparisons, it can be concluded that the thermal material model proposed by Kodur et al. (2020) [27] is preferred to be used to simulate the thermal material properties for UHSC and HSC (having \( f'_c > 65 \) MPa).

### Table 3. Comparison for different models used for simulation of HSC \( (f'_c > 65\text{MPa}) \)

<table>
<thead>
<tr>
<th>Model</th>
<th>Results for ( T_{\text{test}}/T_{\text{FE}} )</th>
<th>Point 1</th>
<th>Point 2</th>
<th>Point 3</th>
<th>Point 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kodur and Sultan (2003) [25]</td>
<td>Mean 0.990</td>
<td>1.271</td>
<td>1.038</td>
<td>0.672</td>
<td></td>
</tr>
<tr>
<td>Kodur and Khaliq (2011) [26]</td>
<td>Mean 0.990</td>
<td>1.270</td>
<td>1.025</td>
<td>0.693</td>
<td></td>
</tr>
<tr>
<td>Kodur et al. (2020) [27]</td>
<td>Mean 0.990</td>
<td>1.280</td>
<td>1.058</td>
<td>0.822</td>
<td></td>
</tr>
</tbody>
</table>

For the reliability of the proposed FE models, Figure 8 shows the correlation between the experimental results and the results obtained by the FE models presented in this study. Figure 8 demonstrates the comparisons with the specimens infilled with HSC, UHSC, and LWC. Thus, it can be concluded that the proposed FE models can precisely predict the temperature developments within the CFDST columns under fire.

![Figure 8. Comparisons between the test results and the FE model predictions for HSC and LWC.](image-url)
4. STRESS ANALYSIS AND DESIGN GUIDELINES

After validating and verifying the proposed FE models for heat transfer analysis of CFDST members under fire, the FE routine has been extended to include the fully coupled thermal-stress analysis. Given the efficiency of the proposed FE routine, it is worthy of use for optimizing the fire design of CFDST members, thereby contributing to the intelligent use of composite structures. The post-fire behaviour of CFDST columns is investigated, and the current design specifications are thoroughly evaluated for the design of such members. To the best knowledge of the authors, current fire standards do not present design models for CFDST columns under fire; besides, limited work is available in this research area. Accordingly, the FE methodology presented in this paper is extensively adopted to provide a database for the axial and flexural capacity of CFDST members under fire. Moreover, an analytical method for predicting the temperature development within the cross-sections is presented to be further used to design such members under fire. New interaction curves considering the material deterioration at high temperatures are of great interest.

5. CONCLUSIONS

With a focus on the heat transfer in UHPC filled double-skin tubes under fire, this paper presents a FE modeling protocol to simulate CFDST for heat transfer analysis under fire. Specifically, the Python scripting technique is utilized for the model generation, job submission, and post-processing of the analysis results. The results are provided through the nodal temperatures versus the fire time (minutes). It is found that various parameters that impact the heat transfer include the thermal properties of materials, the convective heat transfer coefficient, the emissivity coefficient, the thermal contact conductance, and the cross-sectional configurations. All these influencing parameters are incorporated in the proposed algorithm. The proposed algorithm is elaborated via a flowchart, and extensive fire test results collected from the literature establish its robustness and accuracy.

Furthermore, a new model for the thermal contact conductance for the octagonal and hexagonal profiles is proposed using the interpolation method. Finally, an extensive parametric study is carried out. Based on the results of the parametric study, the following key observations are made:

- When the compressive strength of the infill concrete is greater than 65 MPa (i.e., HSC or UHSC), the thermal material model proposed by Kodur et al. (2020) [51] is preferred. This is inferred from the observed improvement in agreement with the experimental fire testing results.
- A combination of the ASCE model for the specific heat and the EC4 model for the thermal conductivity of LWC is recommended for simulating LWC. However, more experimental tests need to be performed on CFDST with LWC as infill material.
- Adopting the NSC for the outer concrete ring and the UHSC for the inner concrete core for relatively smaller hollowness ratios (e.g., 0.4-0.5) leads to better performance in the temperature development simulation within the cross-section.

REFERENCES


These proceedings contain the papers at the TENTH INTERNATIONAL CONFERENCE ON ADVANCES IN STEEL STRUCTURES (ICASS 2020) held in Chengdu, China, from 21 to 23 August 2022. The international conference series on Advances in Steel Structures was initiated in 1996 under the support of The Hong Kong Polytechnic University, which remains very active in fostering its continuation - joined a few years later by the Hong Kong Institute of Steel Construction.

These proceedings bring together most recent findings in numerical, theoretical and experimental research, as well as its practical implementation in design practice in the areas of Assembled Structure, Bridge, Structure Systems, Composite, Design & Analysis, Direct Analysis, New Material, Fatigue, Cold-formed Steel, Intelligent Construction, Seismic Resistance, Green Construction, Corrosion, Fracture, Collapse, Fire, High-Strength Steel, Stability, Stainless Steel, Testing & Monitoring, Impact and Protection, The papers presented in these proceedings come from a wide range of countries/regions and will be a great reference source.

**Volume I:**
- Keynotes Lectures
- Assembled Structure
- Bridge
- Cold-Formed
- Composite
- Connections
- Corrosion, Fracture & Collapse
- Design & Analysis
- Direct Analysis
- Fatigue

**Volume II:**
- Fire
- High-Strength Steel
- Impact and Protection
- Intelligent Construction
- New Materials
- Seismic Resistance
- Stability
- Stainless Steel
- Structure Systems
- Testing & Monitoring