

Article

Experimental Investigation on the Post-Fire Performance of Multiple-Strength-Grade Steel Wires

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Abstract

This study assessed the critical fire resistance of bridge cables by investigating post-fire tensile degradation of high-strength steel wires (1860, 1960, 2100 MPa) heated under initial stress (10% and 40% of design strength) to 300 °C, 400 °C, 500 °C, and 600 °C followed by cooling to an ambient temperature. Tensile tests determined reduction coefficients (RCs) for proportional limit (σ_p), yield strength (σ_y), ultimate strength (σ_u), elastic modulus (E), and elongation (δ) relative to ambient values, with constitutive models for σ_p , σ_y , and σ_u RCs integrating temperature, stress, and grade. Visual observation showed intensified wire hue and reduced necking with increasing temperature. All RCs remained stable up to 300 °C; beyond this temperature, σ_p , σ_y , and σ_u RCs progressively decreased, averaging 77.5%, 65.7%, and 61.9% at 600 °C under lower-level initial stress (10%) and 74.1%, 63.8%, and 60.6% at 400 °C under higher-level initial stress (40%). Elastic modulus exhibited minimal variation, whereas elongation reached its minimum at 400 °C under lower-level initial stress but its maximum at 400 °C under higher-level initial stress. The impact of lower-level initial stress on mechanical properties was negligible, showing a less than 3.2% average RC decrease at 600 °C. Higher-level initial stress accelerated mechanical property degradation with increasing temperature, with comparable degradation patterns observed across different steel wire strength grades. The models confirm maximum temperature dominance in degradation, supporting a recommended critical fire-resistant temperature of approximately 400 °C for bridge cable wires.

Keywords: high-strength steel wires; tensile test; mechanical properties; post-fire performance; elevated temperature



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1. Introduction

As traffic volume increases, so does the number of vehicles transporting flammable and explosive hazardous materials [1,2], thereby heightening the risk of bridge fires due to vehicle collisions and tanker accidents. U.S. highway statistics indicate that there are 376,000 fires each year, resulting in 570 fatalities and economic losses amounting to \$1.28 billion [3], which underscores the significant threat that bridge fires pose to infrastructure and public safety. Notably, bridge fire protection is less adequately addressed in current design codes and maintenance guidelines compared to natural disasters [4–6]. Despite advancements in wind and seismic resistance, fire prevention for bridges has been neglected. Research indicates that fires cause more damage to bridges than earthquakes,

emphasizing the necessity of enhancing fire protection measures [7]. Fires pose a serious threat to the steel components of bridges. High temperatures can soften steel and diminish its strength [8]. When the yield strength decreases, localized failures may occur, potentially leading to bridge collapse. Since bridge cables and high-strength steel wires are critical load-bearing elements, it is essential to study their behavior under fire conditions and to improve their fire resistance [9,10].

To simulate and evaluate temperature fields, researchers developed an enhanced Computational Fluid Dynamics (CFD) model integrated with InfoGAN technology [11,12]. This model considers heat source release and fluid flow conductivity, predicting internal furnace temperatures from partial operational data. However, its practical use is limited by a high computational burden and slow convergence, hindering real-time acquisition of temperature fields. Regarding the degradation mechanisms of high-strength steel wires' load-bearing capacity, prediction methods based on 3D measurement data were proposed [13,14]. These methods incorporate temperature, creep, and corrosion effects on steel wires to predict degradation and establish constitutive models but ignore stress factors, limiting their universality. To accurately assess cable structures' thermal response, theoretical solutions and finite element simulations calculate steel wire temperature distributions; for computational efficiency, 2D thermal response models simulate heat conduction and temperature profiles in cable sections [15–17].

Cable-supported bridges rely mainly on cables as load-bearing components [18]. Without protection, cables are vulnerable to mechanical degradation or failure in fires due to steel wire property deterioration [19], making studying high-temperature mechanical degradation of steel wires crucial for evaluating bridge cable performance during and after fires. Researchers conducted post-fire degradation studies on multiple-strength-grade steel wires (1570 MPa, 1670 MPa, 1770 MPa, 1860 MPa, and 1980 MPa) [20–28]. These studies involved heating experiments and ambient-temperature tensile tests. Currently, 300 °C is recognized as the bridge cable failure threshold [29], adopted as the critical control temperature in technical codes [30]. However, this threshold primarily reflects the damage temperature of the cable's protective layer rather than the performance degradation temperature of the steel wires themselves. The 300 °C threshold also fails to accurately characterize the performance damage temperature of high-strength steel wires, thus creating an urgent need to fill the knowledge gap in the field of their fire performance.

Notably, significant degradation occurs above 400 °C under conditions without initial stress [31,32]. Studies have shown that the elastic modulus of steel wires remains almost constant even when exposed to temperatures up to 900 °C; however, when temperatures reach 400–800 °C, the reduction rates of their yield strength and ultimate strength can be as high as 40–90% and 35–85%, respectively [33,34]. Current research is limited: it focuses on specific strength grades without establishing universal laws, lacks fire resistance studies on over 2000 MPa steel wires, and lacks performance comparisons with conventional wires, thus failing to support fire safety design and post-fire evaluation.

Beyond temperature, multiple factors have been examined in post-fire mechanical evaluations of steel wires. Studies comparing different heating curves [35], steel wire strength grades, cooling methods [36], and initial stress levels [37] during fire simulation experiments indicate that the maximum temperature reached dominates mechanical degradation, with the heating process itself proving practically irrelevant [38]. Notably, degradation exhibits significant scatter even among wires of identical strength grades subjected to similar heating programs, and no clear correlation emerges between strength grade and the degradation of typical mechanical properties. Sprinkler cooling can considerably influence the extent of performance deterioration in sprinkler systems [36,39,40]. Two post-fire cooling measures, i.e., air and water cooling, have also been introduced in

several investigations. The water-cooling method causes a greater degree of degradation in the strength of the steel wire, and the difference from the air-cooling method can reach 28%. Furthermore, initial stress shows negligible influence on wire rope properties below 450 °C. However, its role becomes dominant above this threshold; at 600 °C, specimens under initial stress exhibit ultimate failure stresses approximately 12% lower than unstressed counterparts [41].

Previous studies mainly focused on individual factors (e.g., high temperature or stress) influencing the post-fire mechanical degradation of steel wires. However, comprehensive investigations into their combined effects are limited. Existing research lacks a systematic quantification of how the combined action of high temperature and stress affects the mechanical properties of steel wires after a fire, suggesting further exploration is needed to understand this complex interaction mechanism. To address these gaps, this study tested 135 steel wires (1860–2100 MPa) to evaluate how elevated temperatures and initial stress impact post-fire appearance, failure modes, and key mechanical properties of steel wires under various stress states. Based on experimental results, we established reduction coefficient rules for mechanical property degradation during and after high-temperature exposure. We further developed empirical equations for the high-temperature stress–strain behavior, established a damage model applicable to steel wires of multiple strength grades, and clarified critical temperature thresholds for the fire safety design of high-strength bridge cables, providing recommended indicators for the performance evaluation of bridge cables post-fire.

2. Test Scheme

2.1. Test Specimens

The test specimens comprised high-strength steel wires graded at 1860 MPa, 1960 MPa, and 2100 MPa. All specimens featured identical dimensions: 600 mm length and 7 mm diameter (conforming to GB/T 228.2 [42]), as illustrated in Figure 1. The specimens were produced and processed through the same manufacturing line with the chemical compositions provided in Table 1. Specimens were categorized into three groups by strength grade, each group included five heating conditions corresponding to target maximum temperatures. To mitigate material randomness, high-strength steel wires from the same batch were randomly assigned to different experimental groups, with each initial stress test group comprising 15 specimens as detailed in the experimental matrix presented in Table 2.



Figure 1. Dimensions of test specimen.

Table 1. Chemical composition of the steel wire (%).

Ingredients	C	Mn	Si	S	P	Cu
Atomic ratio	0.75–0.85	0.6–0.9	0.12–0.32	<0.025	<0.025	<0.200

Table 2. Groups of specimens.

Group Name	Grade (MPa)	Initial Stress (MPa)	The Maximum Temperature					Cooling Method	Total Number
			20 °C	300 °C	400 °C	500 °C	600 °C		
A	1860	0	3	3	3	3	3	Nature	15
		186	3	3	3	3	3	Nature	15

Table 2. Cont.

Group Name	Grade (MPa)	Initial Stress (MPa)	The Maximum Temperature					Cooling Method	Total Number
			20 °C	300 °C	400 °C	500 °C	600 °C		
B	1960	0	3	3	3	3	3	Nature	15
		196	3	3	3	3	3	Nature	15
		784	5	5	5	/	/	Nature	15
		784	5	5	5	/	/	Water	15
C	2100	0	3	3	3	3	3	Nature	15
		210	3	3	3	3	3	Nature	15
		840	5	5	5	/	/	Nature	15

2.2. Test Setup

A thermo-mechanical coupling test setup, shown in Figure 2, simulated the actual state of steel wires under fire. Each specimen was rigidly fixed at one end to a steel reaction frame with its opposite end connected to a mechanical loading system. This system applied calibrated tensile forces using a lever principle through adjustable combinations of lever arm lengths and mass blocks. Specimens were heated within an electric high-temperature furnace featuring a cylindrical configuration, a central through-hole, 240 mm diameter, 460 mm height, and maximum operating temperature of 1200 °C.

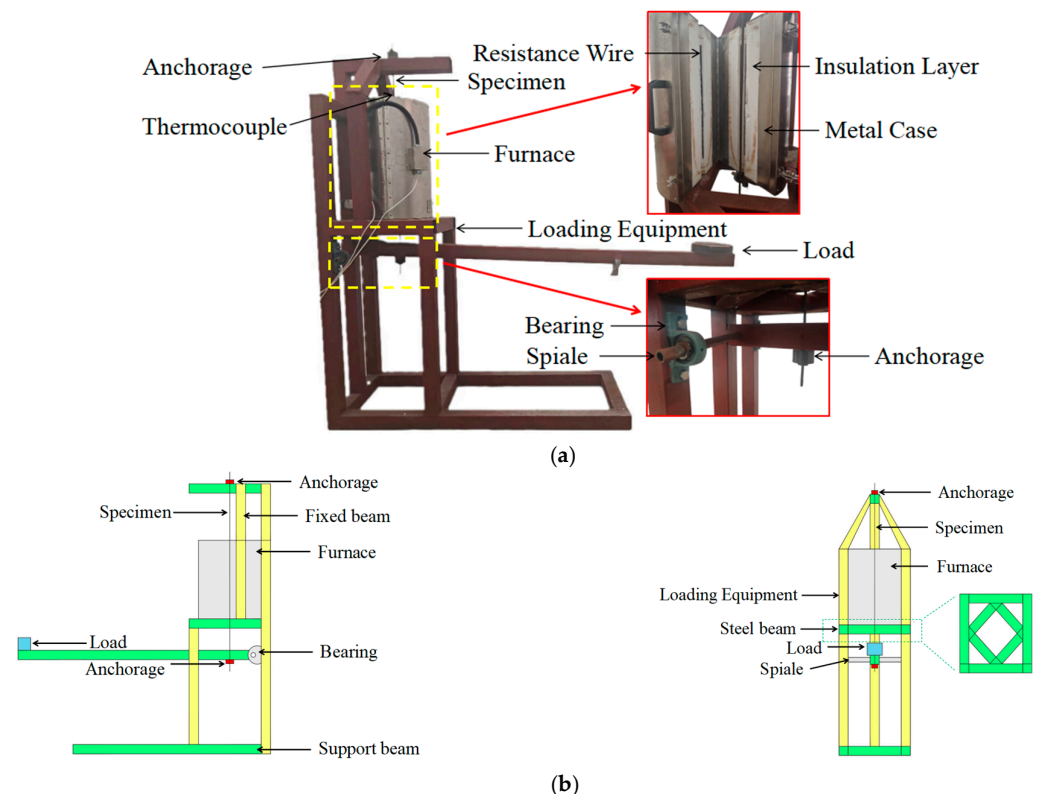


Figure 2. Thermo-mechanical coupling test setup. (a) Device picture; (b) schematic picture.

Following the initial stress application, heating to the target temperature for 30 min, and subsequent cooling to an ambient temperature, tensile testing was performed using an electronic universal testing machine, as shown in Figure 3, with a 100 kN maximum capacity. This machine provides load measurement accuracy of ± 0.1 kN and a measurement range

spanning 0.2–100% of capacity with errors not exceeding $\pm 0.5\%$, while the deformation measurement exhibits errors below 0.5% of the indicated value.

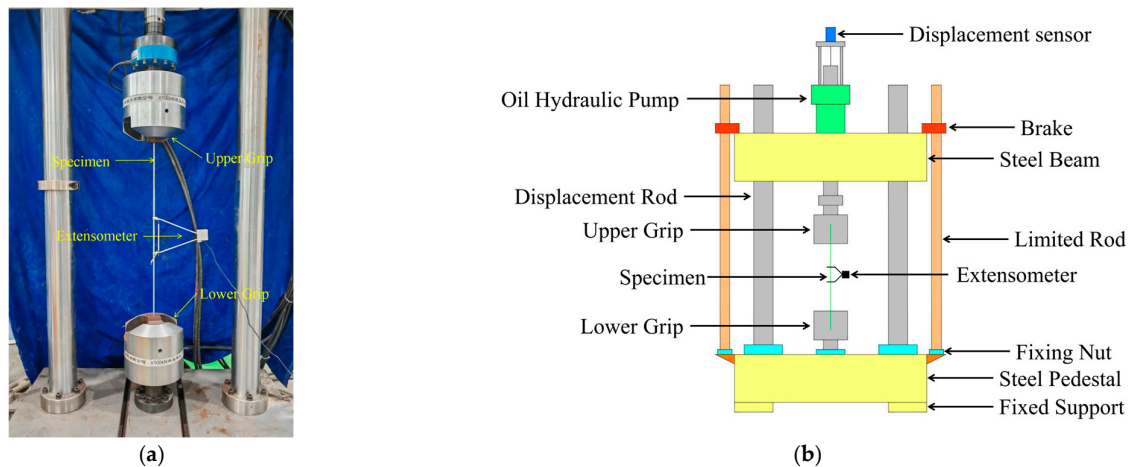


Figure 3. Tensile test setup. (a) Device picture; (b) schematic picture.

Tensile testing proceeded at a crossbeam displacement velocity of 5 mm/min, corresponding to a strain rate of $1.39 \times 10^{-4} \text{ s}^{-1}$ compliant with ISO and ASTM standards. Strain gauges and an extensometer acquired displacement data at 50 samples per second, with all data automatically documented to generate load–deformation curves.

2.3. Test Process

Figure 4 presents the entire test procedure flow. A thermo-mechanical coupling test was performed on steel wire specimens under initial stresses set at a lower level (10% of design stress) and higher level (40% of design stress). Heating progressed per the ISO-834 recommended curve [43,44] to the maximum temperature shown in Figure 5 and was maintained for 30 min. Specimens were then removed from the furnace, cooled to an ambient temperature, and tensile tested at 5 mm/min until fracture failure [42], with load and deformation measured throughout. Machine-recorded force data and extensometer-corrected deformation data yielded load–deformation curves for subsequent analysis. Engineering stress–strain curves derived from this data provided mechanical indicators, including the proportional limit (σ_p), the yield strength (σ_y), the ultimate strength (σ_u), the elastic modulus (E), and the elongation (δ).

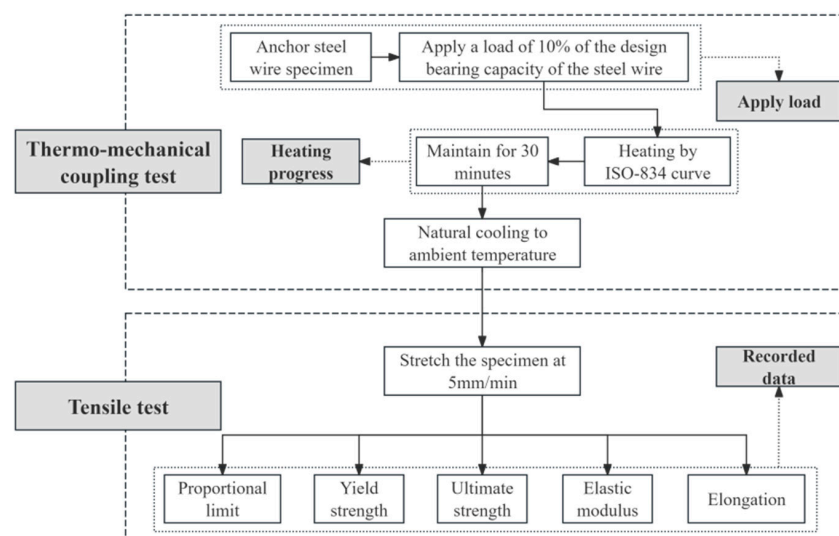


Figure 4. Experimental flowchart.

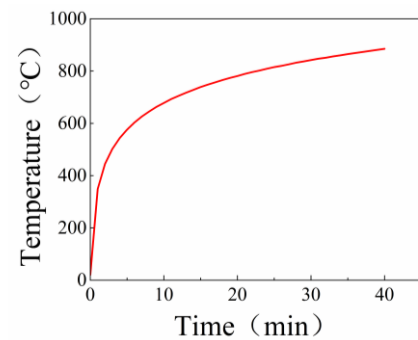


Figure 5. Heating curve.

3. Experimental Results

3.1. Failure Modes and Visual Observation

Following thermo-mechanical coupling tests and subsequent cooling, stretched steel wire specimens exhibited distinct visual appearances corresponding to maximum temperatures, with varying fracture geometries shown in Figure 6. Fracture surface coloration evolved with increasing maximum temperature: specimens tested statically at an ambient temperature displayed gray-white surfaces; at 300 °C surfaces appeared silvery gray; between 400 °C and 500 °C fractures transitioned to bluish gray; at 600 °C surfaces turned charcoal gray. Substantial zinc oxide particle formation indicated galvanized coating deterioration beginning at 500 °C.

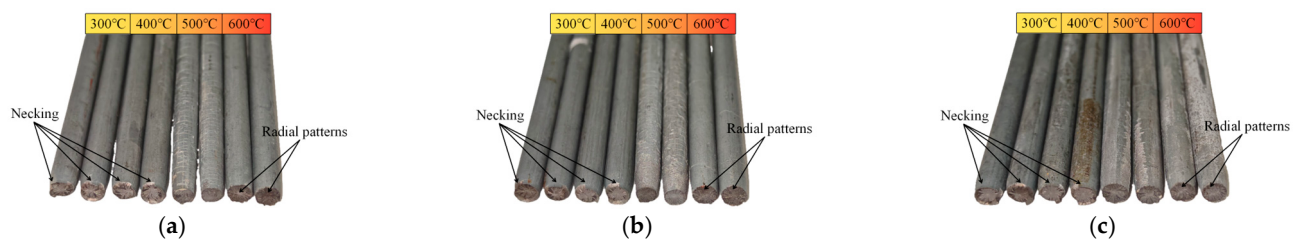


Figure 6. Characterization of high-strength steel wire post tensile test. (a) 1860 MPa, (b) 1960 MPa, (c) 2100 MPa.

The maximum temperature exerted significant influence on fracture morphology, with fracture section shapes under varying maximum temperatures presented in Figures 7 and 8. Specimens without initial stress exhibited irregular, rough fractures with pronounced necking below 400 °C. At 400 °C, reduced cross-sectional area and diminished necking indicated a brittle transition. By 600 °C, fractures became planar with distinct radial patterns, confirming brittle failure. This transition was further shaped by material microstructure and composition, with grain size, precipitates, and alloying elements adjusting the fracture features alongside thermal and stress effects. An initial stress application accelerated this ductile-to-brittle transition.



Figure 7. Fracture morphology of high-strength steel wire. (a) With initial stress; (b) without initial stress.

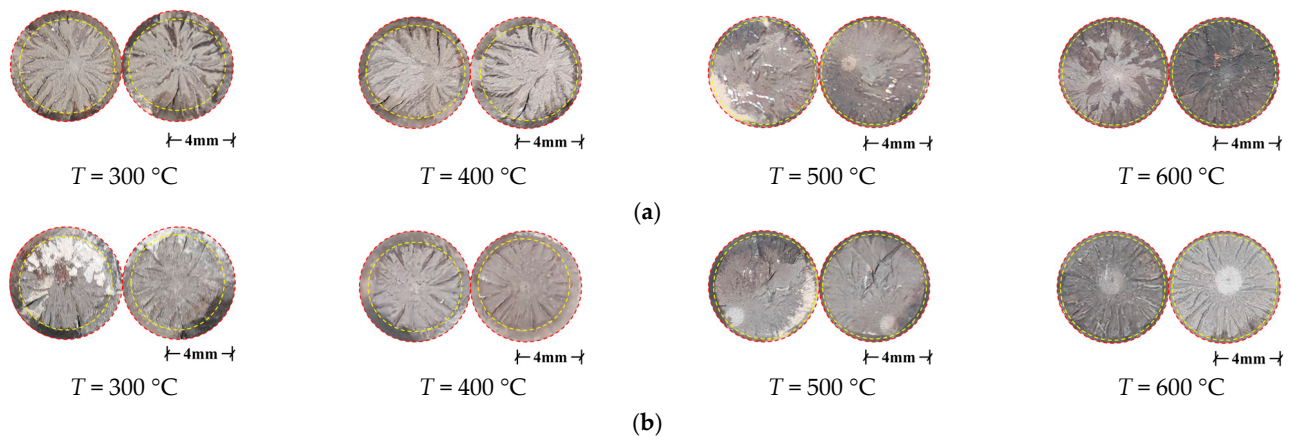


Figure 8. Fracture characteristics of high-strength steel wire post tensile test. (a) With initial stress; (b) without initial stress.

Scanning electron microscopy captured microscopic fracture morphologies across temperature levels, as depicted in Figure 9, comprehensively depicting failure modes. The SEM micrograph of a 600 °C fracture exhibits a planar surface with radial striations. These radial features denote crack propagation from a central origin with minimal plastic deformation, characteristic of brittle fracture. The planar texture and radial markings indicate rapid crack propagation under stress at 600 °C, aligning with brittle failure (little ductile deformation, unlike ductile-featured necked fractures at lower temperatures). Results demonstrated a progressive necking reduction below 400 °C and transition to brittle planar fractures above 600 °C.

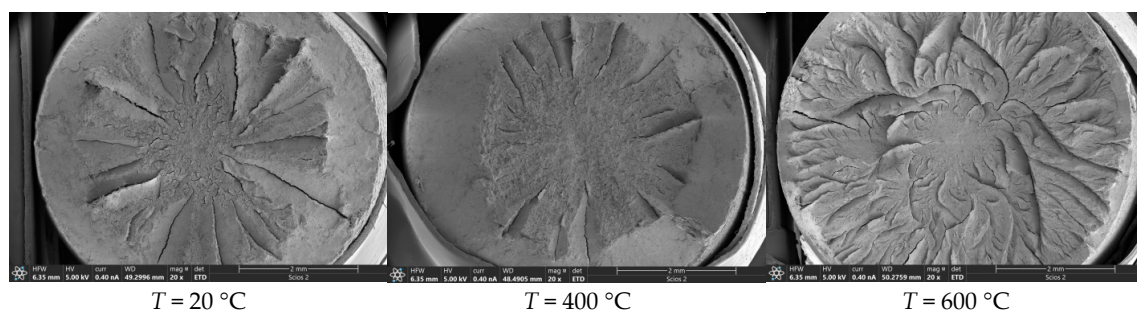


Figure 9. Fracture microstructure of high-strength steel wires at different temperatures.

3.2. Stress-Strain Curves

Tensile testing yielded engineering stress-strain curves for all specimens, as shown in Figure 10, showing similar profiles but significant mechanical indicator variations across test conditions. Below 400 °C, inter-curve deviations remain minimal, and temperatures exceeding 400 °C produced substantial degradation. Yield strength (σ_y) and ultimate strength (σ_u) progressively decreased with rising temperatures while the elastic modulus (E) exhibited minimal variation throughout the heating regime.

3.3. Key Mechanical Performance Indicators

Stress-strain curve analysis extracts key mechanical indicators characterizing steel wire performance employing four parameters—proportional limit (σ_p), yield strength (σ_y), ultimate strength (σ_u), and elastic modulus (E)—to quantify degradation. Figure 11 identifies their characteristic positions: proportional limit denotes the maximum elastic-stage stress, yield strength represents the stress at 0.2% plastic strain (proof stress), and ultimate strength corresponds to peak stress. The elastic modulus is computed as the linear slope between 10% and 50% of the yield strength [45].

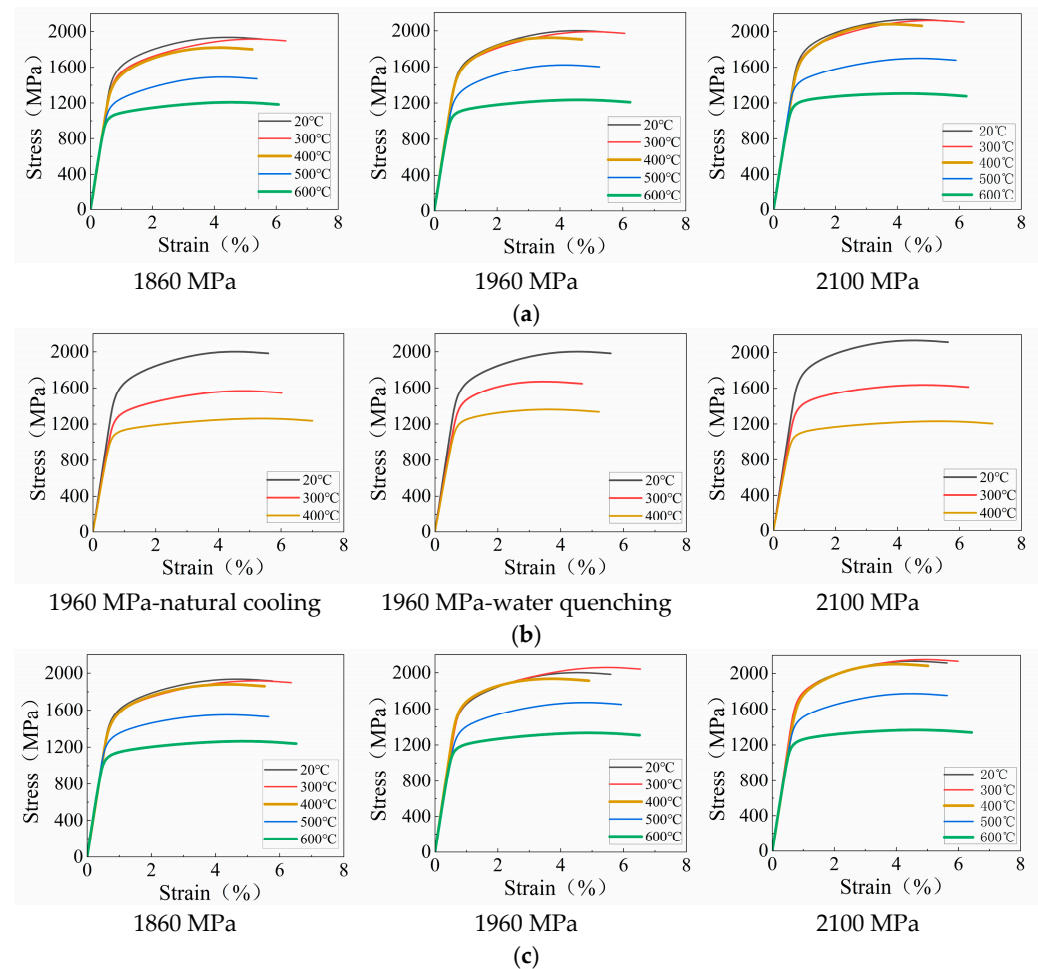


Figure 10. Stress–strain curves of high-strength steel wires. (a) With 10% initial stress, (b) with 40% initial stress, and (c) without initial stress.

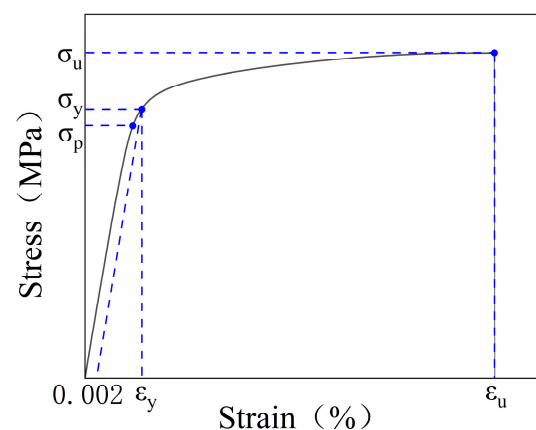


Figure 11. Physical definition of four indicators.

Experimental data processing computed average values for σ_p , σ_y , σ_u , and E across all test cases, which are presented in Tables 3–5, with reduction coefficients standard deviations in Table 6. The standard deviation of the E is so small that it cannot be presented. To analyze the influence mechanisms of different factors, the RCs of four indicators are calculated by dividing by normalizing against ambient-temperature conditions and presented in the table as well. As shown in the tables, three strength indicators, i.e., σ_p , σ_y and σ_u , decrease monotonically as the temperature rises, while the value of E remains almost constant under various elevated temperatures.

Table 3. Mechanical properties of 1860 MPa high-strength steel wires.

Specimen	$T/^{\circ}\text{C}$	σ_p/MPa	σ_p/σ_{p0}	σ_y/MPa	σ_y/σ_{y0}	σ_u/MPa	σ_u/σ_{u0}	E/GPa	E/E_0	$\delta/\%$
Normal temperature	20	1267	1.000	1690	1.000	1967	1.000	228	1.000	5.542
With 10% initial stress	300	1255	0.991	1637	0.969	1948	0.990	217	0.952	6.310
	400	1164	0.919	1569	0.928	1852	0.942	221	0.969	5.226
	500	1051	0.830	1255	0.743	1521	0.773	222	0.974	5.376
	600	995	0.785	1096	0.649	1233	0.627	230	1.009	6.074
	700	/(fracture)								
Without initial stress	300	1281	1.011	1673	0.990	1949	0.991	224	0.982	6.364
	400	1194	0.942	1661	0.983	1910	0.971	219	0.961	5.531
	500	1122	0.886	1384	0.819	1579	0.803	237	1.039	5.657
	600	1031	0.814	1169	0.692	1288	0.655	231	1.013	6.516

Table 4. Mechanical properties of 1960 MPa high-strength steel wires.

Specimen	$T/^{\circ}\text{C}$	σ_p/MPa	σ_p/σ_{p0}	σ_y/MPa	σ_y/σ_{y0}	σ_u/MPa	σ_u/σ_{u0}	E/GPa	E/E_0	$\delta/\%$
Normal temperature	20	1336	1.000	1728	1.000	2033	1.000	230	1.000	5.594
With 10% initial stress	300	1336	1.000	1715	0.992	2022	0.995	228	0.991	6.059
	400	1263	0.945	1660	0.961	1955	0.962	227	0.987	4.699
	500	1134	0.849	1395	0.807	1653	0.813	219	0.952	5.256
	600	1027	0.769	1143	0.661	1259	0.619	222	0.965	6.231
	700	/(fracture)								
With 40% initial stress	300	1153	0.863	1375	0.796	1596	0.785	204	0.887	6.021
	400	1046	0.783	1160	0.671	1286	0.633	199	0.865	6.998
	500	/(fracture)								
With 40% initial stress (splash water cooling)	300	1247	0.933	1497	0.866	1699	0.836	201	0.874	4.681
	400	1121	0.839	1275	0.738	1384	0.681	193	0.839	5.219
	500	/(fracture)								
Without initial stress	300	1403	1.050	1778	1.029	2090	1.028	222	0.965	6.535
	400	1323	0.990	1737	1.005	1963	0.966	236	1.026	4.899
	500	1186	0.888	1455	0.842	1702	0.837	221	0.961	5.929
	600	1099	0.823	1232	0.713	1358	0.668	226	0.983	6.513

Table 5. Mechanical properties of 2100 MPa high-strength steel wires.

Specimen	$T/^{\circ}\text{C}$	σ_p/MPa	σ_p/σ_{p0}	σ_y/MPa	σ_y/σ_{y0}	σ_u/MPa	σ_u/σ_{u0}	E/GPa	E/E_0	$\delta/\%$
Normal temperature	20	1461	1.000	1893	1.000	2166	1.000	228	1.000	5.633
With 10% initial stress	300	1470	1.006	1862	0.984	2155	0.995	223	0.978	6.150
	400	1435	0.982	1781	0.941	2114	0.976	221	0.969	4.790
	500	1300	0.890	1497	0.791	1731	0.799	237	1.039	5.892
	600	1126	0.771	1253	0.662	1326	0.612	221	0.969	6.224
	700	/(fracture)								
With 40% initial stress	300	1236	0.846	1505	0.795	1665	0.769	206	0.904	6.291
	400	1020	0.698	1144	0.604	1254	0.579	194	0.851	7.070
	500	/(fracture)								
Without initial stress	300	1524	1.043	1889	0.998	2185	1.009	239	1.048	5.988
	400	1474	1.009	1839	0.971	2134	0.985	220	0.965	5.018
	500	1333	0.912	1551	0.819	1805	0.833	223	0.978	5.636
	600	1152	0.789	1293	0.683	1389	0.641	226	0.991	6.427

Table 6. Standard deviation of reduction coefficients for various mechanical property indicators.

Specimen	$T/^{\circ}\text{C}$	1860 MPa				1960 MPa				2100 MPa			
		σ_p	σ_y	σ_u	δ	σ_p	σ_y	σ_u	δ	σ_p	σ_y	σ_u	δ
Normal temperature	20	0.007	0.004	0.011	0.058	0.011	0.013	0.008	0.026	0.006	0.009	0.007	0.029
With 10% initial stress	300	0.014	0.017	0.010	0.062	0.009	0.012	0.014	0.047	0.011	0.016	0.012	0.043
	400	0.020	0.022	0.017	0.053	0.023	0.025	0.020	0.039	0.024	0.027	0.024	0.036
	500	0.032	0.019	0.015	0.034	0.015	0.011	0.028	0.027	0.006	0.025	0.006	0.040
	600	0.014	0.022	0.025	0.047	0.021	0.024	0.013	0.031	0.022	0.026	0.025	0.020
With 40% initial stress	300	/	/	/	/	0.021	0.025	0.027	0.043	0.026	0.019	0.024	0.017
	400	/	/	/	/	0.036	0.027	0.038	0.031	0.042	0.025	0.036	0.039
With 40% initial stress (splash water cooling)	300	/	/	/	/	0.027	0.012	0.018	0.061	/	/	/	/
	400	/	/	/	/	0.029	0.050	0.018	0.057	/	/	/	/
Without initial stress	300	0.024	0.007	0.012	0.052	0.021	0.013	0.018	0.037	0.027	0.035	0.016	0.039
	400	0.032	0.026	0.021	0.060	0.013	0.039	0.019	0.056	0.027	0.021	0.018	0.052
	500	0.016	0.029	0.029	0.040	0.030	0.044	0.048	0.055	0.027	0.032	0.033	0.041
	600	0.024	0.007	0.012	0.052	0.021	0.013	0.018	0.037	0.027	0.035	0.016	0.039

Furthermore, specimens subjected to higher-level initial stress (40%) exhibited cross-sectional contraction during heating, reducing actual cross-sectional area; subsequent tensile tests using the original area for E calculations consequently yielded underestimated values. At lower-level initial stress (10%), δ minimized after 400 °C heating, whereas higher-level initial stress resulted in gradual δ increase.

4. Impact Analysis

4.1. Temperatures

Building upon elastic modulus (E) reduction under higher-level initial stress (40%), stress levels below a lower-level (10%) E exhibit minimal variation, remaining effectively independent of temperature, initial stress, and strength grade with reduction coefficient fluctuations below a 4.8% range. Consequently, subsequent analysis focuses exclusively on the remaining four indicators. Specimens are grouped by strength grade (1860, 1960, and 2100 MPa) in Figure 12, with the reduction coefficients (RCs) column demonstrating a temperature invariance below 300 °C (all RCs within 95–105%); beyond 300 °C each indicator follows distinct thermal degradation behaviors.

Progressive temperature increases above 300 °C drive monotonic declines in σ_p , σ_y , and σ_u . At 600 °C, average RCs decrease to 80.9%, 69.6%, and 65.5%, respectively. Under lower-level initial stress, these values further reduce to 77.5%, 65.7%, and 61.9%, with specimens fracturing before reaching 700 °C. Higher-level initial stress accelerates damage precipitously—inducing fracture before 500 °C. At 400 °C under higher-level initial stress, RCs plummet to 74.1%, 63.8%, and 60.6%, establishing 400 °C as the critical threshold where lower-level initial stress conditions preserve strength integrity while high stress precipitates fracture. Elongation (δ) displays non-monotonic behavior: under lower-level initial stress conditions, δ minimizes at 400 °C, averaging 87.7% of ambient-temperature values, whereas higher-level initial conditions produce continuous δ increases with temperature. Compared to primary mechanical indicators, δ demonstrates larger standard deviations showing significant numerical fluctuations. Evaluation criteria for fire-damaged cable ductility should therefore adopt conservative thresholds to prevent overestimation due to elongation variability.

Setting this 400 °C critical temperature establishes safety design indicators for post-fire bridge performance assessment. Monitoring bridge cable stress enables determination

of post-fire safety performance through combined analysis of temperature exposure and stress history.

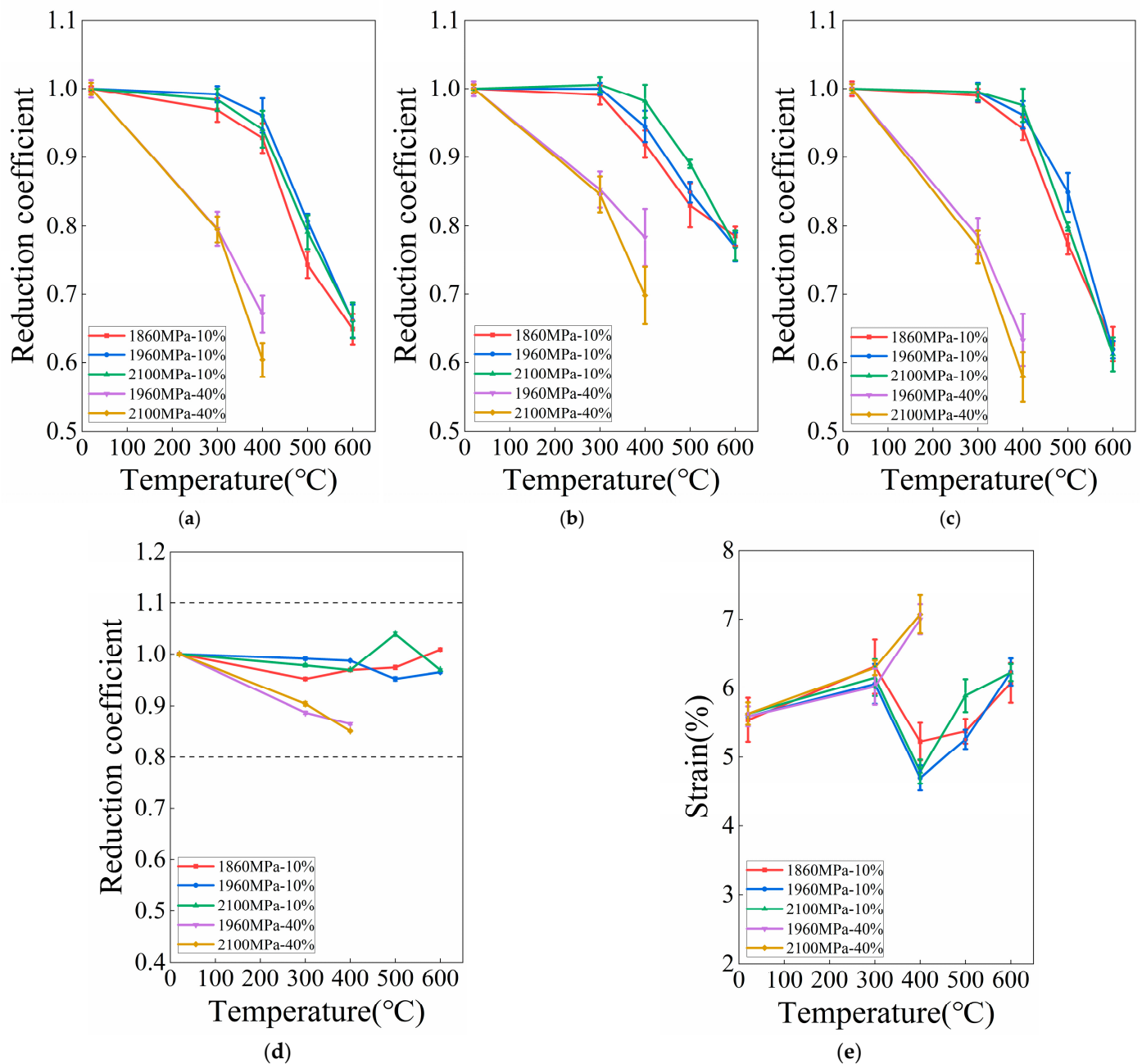


Figure 12. Scatters of σ_p , σ_y , σ_u , E , and δ with temperatures. (a) Proportional limit, (b) yield strength, (c) ultimate strength, (d) elastic modulus, and (e) elongation.

4.2. Initial Wire Stress

Experimental results, as shown in Tables 3–5, indicate that initial stress exerts a weak influence on E and δ but moderate impact on three strength indicators. Figure 13 illustrates temperature-dependent variation trends of these strength indicators grouped by initial stress magnitude. Lower-level initial stress (10%) decreases less than 7.6% across temperatures with average reduction coefficients of 3.2%. The indicator decreases by a value of 1.8–5.4% at 600 °C, establishing its negligible effect on mechanical performance. Higher-level initial stress (40%) substantially accelerates degradation: strength reduction coefficients at 300 °C approximate those at 500 °C under lower-level initial stress with

parallel failure progression. Fracture occurs above 600 °C for lower-level initial stress specimens versus above 400 °C for higher-level initial stress counterparts.

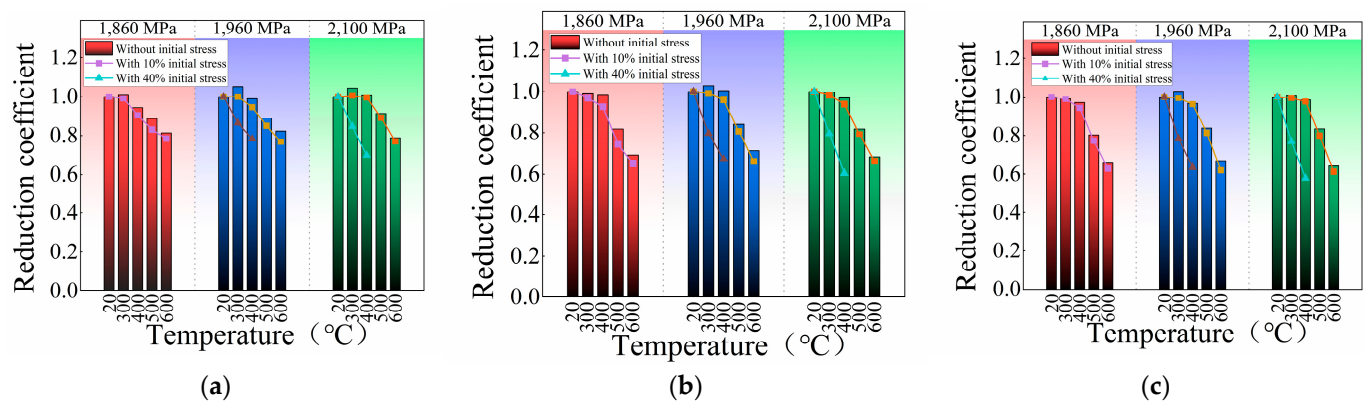


Figure 13. The influence of initial stress on the mechanical properties of steel wires with different strength-grades. (a) Proportional limit, (b) yield strength, and (c) ultimate strength.

These distinct degradation behaviors can be attributed to temperature–stress coupling effects on dislocation activity. At elevated temperatures, thermal energy facilitates dislocation migration, while initial stress provides an additional driving force for dislocation motion. For lower-level initial stress, the combined energy is insufficient to overcome lattice friction and grain boundary pinning, resulting in slow dislocation accumulation and minimal strength reduction. In contrast, higher-level initial stress lowers the activation barrier for dislocation slip and climb, accelerating cross-boundary movement and promoting microcrack formation at lower temperatures. This explains why specimens under 40% initial stress exhibit earlier fracture (above 400 °C) and more rapid strength degradation compared to those under 10% initial stress.

4.3. Strength Grade of Steel Wires

Figure 13 reveals discernible variations in post-temperature strength indicators across high-strength steel wire grades, with consistent maximum inter-grade strength differentials of approximately 5.0%. This disparity extends to high-temperature mechanical properties, including proportional limit, yield strength, and ultimate strength, indicating that commonly employed bridge suspension wires exhibit similar failure mode characteristics and mechanical performance reduction patterns. Their temperature response primarily manifests above the critical temperature; increased strength grades show no significant elongation enhancement, maintain comparable failure characteristics, and demonstrate minimal elastic modulus variation.

4.4. Cooling Method

To investigate the influence of cooling methods on the mechanical properties of high-strength steel wires, the experiment compared the mechanical properties and RCs obtained from tensile tests of 1960 MPa high-strength steel wire specimens under higher-level initial stress (40%) after being heated to 300 °C and 400 °C, followed by natural cooling and water quenching, respectively. Figure 14 shows water quenching increased σ_p , σ_y , and σ_u by 6.3%, 6.9%, and 5.0%, respectively, but caused marked elongation reduction. Microstructurally, rapid cooling triggers non-diffusive austenite transformation, forming hard brittle martensite or bainite structures, explaining water quenching's strength enhancement at the expense of toughness relative to natural cooling.

To further explore the microstructural basis and practical relevance, an additional SEM microscopic analysis was conducted on water-quenched specimens, as shown in

Figure 15. The images clearly reveal a network of interconnected cracks: primary reticulated cracks span the microstructure in crisscrossing patterns, with finer linear branch cracks propagating perpendicularly from their main trunks. These crack networks, formed by rapid cooling-induced thermal stress concentration, exhibit uneven distribution—denser in regions with higher thermal gradient effects. Notably, water quenching, with rapid cooling from direct water contact, aligns closely with fire sprinkler cooling in core mechanisms: both use water-mediated heat extraction for phase transformations. Though cooling rates differ, with water quenching at 10^2 – 10^3 °C/s and sprinklers at 10^0 – 10^1 °C/s, the stress-driven crack formation mechanisms were parallel. Such crack characteristics under water quenching largely represent those under real fire spray cooling, aiding understanding of material integrity under thermal-mechanical loading.

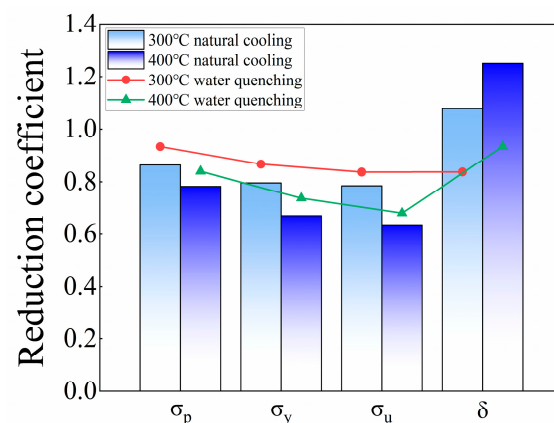


Figure 14. The influence of natural cooling and water quenching on the mechanical properties of steel wire.

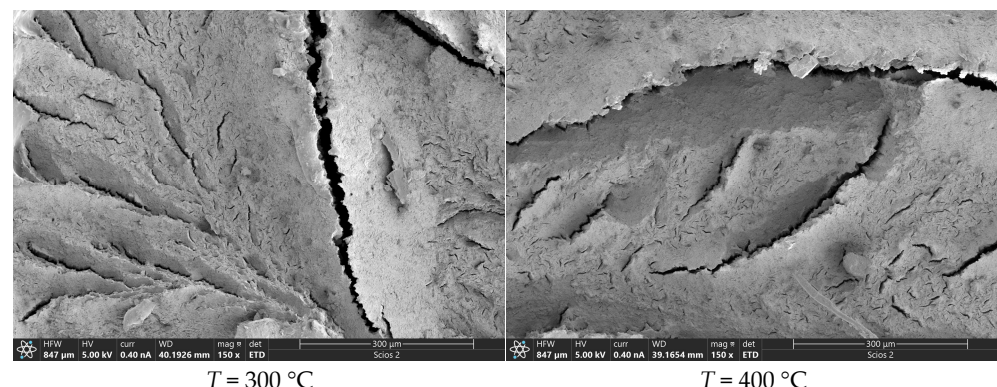


Figure 15. Fracture microstructure of high-strength steel wire post water quenching.

5. Constitutive Model for Reduction Coefficients

Given elastic modulus reduction's independence from temperature, initial stress, and strength grade coupled with significant elongation fluctuation, the post high-temperature mechanical property reduction model focuses exclusively on strength indicators σ_p , σ_y , and σ_u . Based on physical mechanisms and data characteristics, the least squares method parameter estimation produces the empirical Equation (1).

Building on Section 4.3's analysis demonstrating similar degradation patterns across strength grades, a unified empirical equation incorporating 0–100% initial stress and temperature variation is established. Model validation against experimental data appears in Figure 16.

Additionally, by comparing the reduction patterns of proportional limit, yield strength, and ultimate strength reported in the reference literature and referencing them with the

experimental results from [46–48], as shown in Figure 17, it is evident that the calculation outcomes of the aforementioned references are largely consistent with those of this study, thereby validating the accuracy of the present results.

$$\begin{aligned} RC_{\sigma_p} &= 0.996 + 4.577 \times 10^{-4}T + 3.905 \times 10^{-4}\sigma - 1.183 \times 10^{-6}T^2 - 1.368 \times 10^{-3}T \cdot \sigma - 1.803 \times 10^{-2}\sigma^2 \\ RC_{\sigma_y} &= 0.998 + 7.683 \times 10^{-4}T + 1.900 \times 10^{-2}\sigma - 2.081 \times 10^{-6}T^2 - 1.896 \times 10^{-3}T \cdot \sigma - 1.946 \times 10^{-3}\sigma^2 \\ RC_{\sigma_u} &= 0.996 + 7.353 \times 10^{-4}T + 2.136 \times 10^{-2}\sigma - 1.814 \times 10^{-6}T^2 - 2.277 \times 10^{-3}T \cdot \sigma - 2.136 \times 10^{-3}\sigma^2 \end{aligned} \quad (1)$$

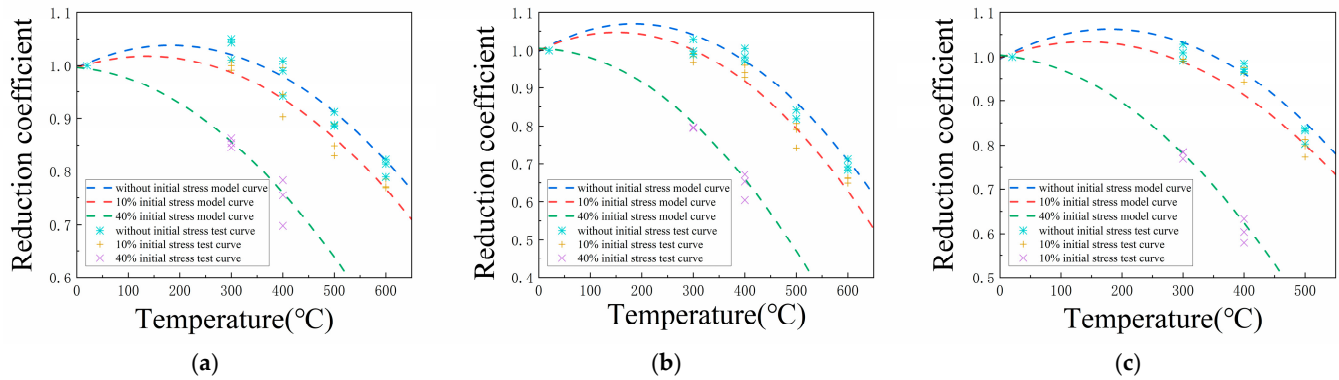


Figure 16. Comparison of performance reduction coefficient curves for high-strength steel wires post high-temperature cooling. (a) Proportional limit, (b) yield strength, and (c) ultimate strength.

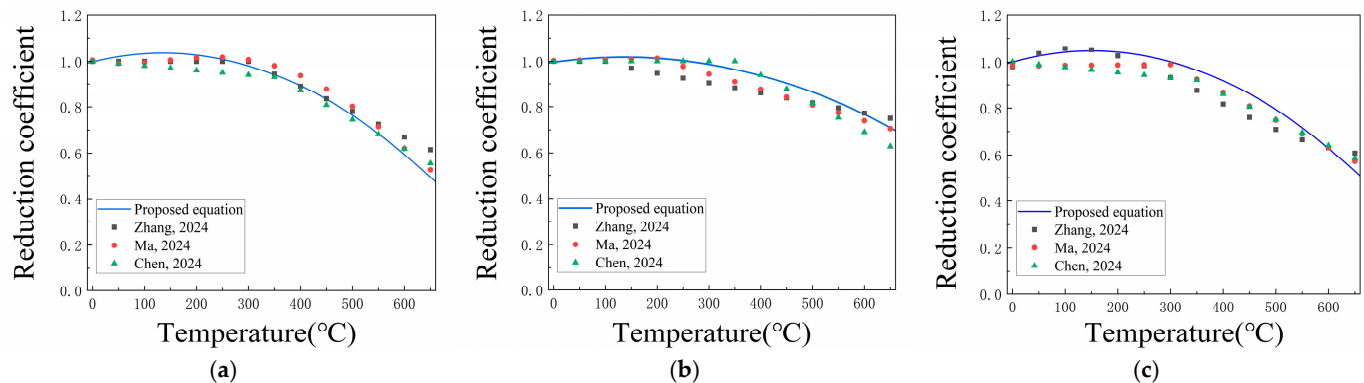


Figure 17. Comparison of performance reduction coefficient models. (a) Proportional limit, (b) yield strength, and (c) ultimate strength [46–48].

6. Conclusions

To evaluate the post-fire degradation mechanism of bridge steel wires, a thermo-mechanical coupling test was carried out on high-strength steel wires. The test involved different elevated temperatures and initial stress levels, which reached up to 10% and 40% of the design strength, respectively. After acquiring the engineering stress–strain curves of the heated steel wires, five mechanical indicators were extracted to systematically explore the combined influences of temperature, initial stress, and strength grade. Through careful examination and analysis, the following key conclusions were derived:

1. The transition of the fracture mode is critically contingent upon the maximum temperature. When the temperature is below 300 °C, fractures display irregular roughness accompanied by necking phenomena. At 400 °C, the fracture cross-sections start to contract, and by 600 °C, the fractures transform into planar surfaces with radial patterns, which verifies the ductile-to-brittle transition. The application of initial stress notably expedites this transition process.
2. The evolution of mechanical properties reveals that the reduction coefficients (RCs) of the elastic modulus vary within 4.8%, demonstrating negligible dependence on

- temperature, stress, and strength grade. Conversely, strength indicators such as the proportional limit (σ_p), yield strength (σ_y), and ultimate tensile strength (σ_u) exhibit significant decline. Specifically, these values decrease notably above 400 °C, reaching approximately 80%, 70%, and 65% of their ambient-temperature counterparts at 600 °C, respectively. When subjected to a lower-level initial stress (10%), the strength of the specimen decreased by only 1.2–7.6% compared with that without initial stress, with an average decrease of 3.2%, and fracture occurred before reaching 700 °C. In contrast, higher-level initial stress (40%) accelerates degradation dramatically, causing specimens to fracture below 500 °C and reducing the RCs of σ_p , σ_y , and σ_u to approximately 75%, 65%, and 60% at 400 °C. The elongation (δ) at break reaches its minimum value (87.7% of ambient temperature) at 400 °C under lower-level initial stress conditions but increases steadily under higher-level initial stress scenarios. Consequently, 400 °C is identified as the critical temperature threshold for mechanical property degradation. Setting the critical temperature establishes safety design indicators for evaluating post-fire bridge safety performance. Additionally, water quenching enhances strength properties but compromises toughness relative to natural cooling.
3. In analyzing the mechanical properties of post-fire steel wires, this study focuses on the reduction trends of three primary performance indicators. By comprehensively considering the initial stress level and temperature change dynamics, a curve-fitting approach was employed to establish the empirical equation for the reduction coefficient of post-fire steel wires. A damage model applicable to steel wires of multiple strength grades was developed. This model has demonstrated good effectiveness in evaluating the degradation law and evolution mechanism of the mechanical properties of steel wires under fire, providing strong theoretical support and practical tools for the safety performance analysis of bridge cable after fire.

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