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Zhi-Xiang Yu

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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.
ANALYSIS OF TRANSIENT STRUCTURAL RESPONSES OF STEEL FRAMES WITH NONSYMMETRIC SECTIONS UNDER EARTHQUAKE MOTION (ICASS’2020)

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Abstract: An accurate structural analysis is a fundamental requirement for modern design. Nevertheless, this is often difficult for systems comprised of nonsymmetric members, primarily because of their complicated cross-section shapes with non-coincidence of shear center and centroid and complex buckling modes. Recent research using efficient line-finite-element formulations has made significant progress in simulating the buckling behavior of arbitrary open-section members for static loads. This paper extends this method by providing for second-order dynamic analysis of nonsymmetric sectional members. The numerical algorithms, including mathematical derivations, are provided and thoroughly validated via their implementation within the nonlinear analysis program MASTAN2-v5.

Keywords: Nonlinear Analysis; Dynamic; Numerical Algorithms; Warping

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1 INTRODUCTION

Nonsymmetrical thin-walled steel sections, such as those shown in Fig. 1, have high strength-to-weight and stiffness-to-weight ratios, and as a result, they have been used extensively in contemporary structures. The prominent features of such sections include: (a) the shape is often complex making it difficult to calculate its geometrical properties; (b) the shear center (s.c.) and the geometrical centroid (g.c.) are rarely coincident, as shown in Fig. 1, which makes the section likely to twist while bending under load, and (c) the plate material is usually thin and may be susceptible to local and distortional buckling thereby reducing strength and stiffness [1]. As a consequence of these attributes, an accurate prediction of the deflections and buckling strengths of systems constructed of thin-walled steel members [2] is complicated. The recent research by the authors [3, 4], which has contributed to progress on simulating the global buckling behavior of arbitrary open-section members for static loads, is developed and extended for dynamic effects in the present study.

Modern structural design methods, such as the direct analysis method within ANSI/AISC 360-16 [5] and the second-order design approach in Eurocode 3 [6], essentially require a nonlinear analysis to explicitly model the buckling behavior of the system and corresponding members. To meet such a requirement, several numerical solutions are employed for the analysis of thin-walled members, and include a shell finite-element method (SFEM) [7, 8], finite-strip method (FSM) [9, 10], generalized beam theory (GBT) [11] and line finite-element method (LFEM) [12, 13].
In recent years, several beam-column element formulations have been presented that consider the Wagner effects [3, 4] for members with nonsymmetrical open-sections for use in LFEM analyses. For example, Rinchen and Rasmussen [13] propose a force-based element with seven degree-of-freedoms (DOFs) at each of its end-nodes, which allows the offset between the centroid and the shear center, and their routines are available in OpenSEES [14]. Shortly after, Liu et al. [4] independently derived a displacement-based element based on the Updated-Lagrangian (UL) description for arbitrary shapes of open-section members, and this element has been implemented within the educational structural analysis software MASTAN2 [15] version 4.0. Although these two methods can effectively capture the complex global buckling modes of nonsymmetrical-section members under bending, they are sometimes inaccurate when the member is also subject to compression. Later, Liu et al. [3] improved the element formulations for automatically considering the deformation due to compression towards the weakest principal axis, thereby now making the element in MASTAN2 capable of modeling the coupled deformations caused by the Wagner effects regardless of any loading conditions. Although recent research has made important progress in simulating the buckling behavior of arbitrary open-section members for static analysis, there is still limited research on the transient behavior of such members when subject to dynamic excitations. It is with this purpose that this research extends the LFEM for second-order elastic time-history analysis of systems constructed of members with nonsymmetrical cross sections.

Generally, when the cross-section is nonsymmetrical, and the effect of the misalignment of the centroid and the shear center is taken into account, the dynamic response of a member with such a section may be different than that of a member with a doubly symmetric cross section. In the case of monosymmetric section members (Fig. 2), the repeated flexure and twisting about the axis of the member may induce apparent warping and thereby additional twisting
deformations, which can be attributed primarily to the torsional stiffness of the open section being quite small. Thus, this can cast doubt on the accuracy of dynamic analyses that assume member cross-sections are doubly symmetric. Indeed, the dynamic response of members of symmetrical and nonsymmetrical sections may be significantly different, and such differences should be considered in design.

To solve transient problems, such as those induced by earthquakes, the time-history analysis method is first reviewed. The numerical integration algorithm over the time domain uses Newmark’s approach [16], which can be configured to be unconditionally stable and is widely used in structural dynamic analyses. A consistent mass matrix that accurately describes the accelerated motions within the member is assumed [17]. For simulating the damping properties, the Rayleigh damping model [18] is adopted. The numerical procedure for conducting the time-history analysis is presented below.

In summary, this paper will provide an overview of the element formulation for the LFEM employed, and the cross-section analysis algorithm is presented in detail. The direct time-integration approach for transient analysis is explained, and its numerical implementation within the educational software MASTAN2-v5.0 [19] is illustrated. Finally, a series of benchmark examples are provided for demonstrating the accuracy of the proposed method.

2 ASSUMPTIONS

The assumptions made for the numerical method presented in this paper include: (a) Euler-Bernoulli element is valid; (b) material is elastic and homogenous; (c) local and distortional buckling are not considered; (d) large deflection is modeled, but strain is assumed to be small; and (e) all forces are applied at the nodes and are conservative.

3 LINE FINITE-ELEMENT METHOD FORMULATION

3.1 The total potential energy equation

The strain energy $U$ considering the linear and nonlinear parts of the Green-Lagrange strain-tensors was previously formulated by the authors [3, 4]. For the complete potential energy equation and simplified form of the potential strain equation $U$, please refer to literatures [3, 4].

3.2 Formulation of displacement functions

In an FE analysis, the structural system is represented by many elements interconnected at nodes. Concentrated loads and moments are applied only at nodes, where the equilibrium equations are established. Deformations along element lengths are described utilizing shape functions, which can be found in the literature (e.g., Liu and McGuire, et al. [3, 4, 20])

Figure 3. Principal-axes coordinate system and the element local DOFs

Accordingly, and assuming the cross-section does not distort during loading, the displacements at any arbitrary point along the element’s length and within its cross-section,
whose coordinate \((x,v,w)\) is given with respect to the centroid \((v_0,w_0)\), and the location along the element length is \(x\) can be expressed as;

\[
u(x,v,w) = u_0(x) - v \frac{\partial v_0(x)}{\partial x} - w \frac{\partial w_0(x)}{\partial x} - w_0 \frac{\partial \theta(x)}{\partial x} \tag{1}
\]

\[
u(x,v,w) = v_0(x) - (w - w_0) \theta(x) \tag{2}
\]

\[
w(x,v,w) = w_0(x) - (v - v_0) \theta(x) \tag{3}
\]

where, \(v_S\) and \(w_S\) are the coordinates of the shear center with respect to the centroid, and \(\omega_n\) is the normalized unit warping constant.

### 3.3 Element’s tangent stiffness matrix

By taking the second variation of the total potential energy \(\Pi\), the resulting equation is written as,

\[
\delta^2 \Pi = [K]_e \{\Delta u\} - \{\Delta f\} = 0 \tag{4}
\]

in which \(\{\Delta u\}\) and \(\{\Delta f\}\) are the vectors of the incremental nodal displacements and forces respectively, and \([K]_e\) is the element’s tangent-stiffness matrix. This matrix includes the linear elastic \([k_L]\) and the geometric \([k_G]\) stiffness matrices provided by McGuire, et al. [32], and the additional geometric stiffness matrix \([k_U]\), which is provided by Liu, et al. [4] to account for nonsymmetrical section-effects, with

\[
[K]_e = [T][K_L][\tau] + [K_G] + [K_U][T]^T \tag{5}
\]

Where \([\tau]\) is the transformation matrix to account for the section’s principal axes \((x-v-w)\) being inclined by an angle \(\phi\) (Fig. 3) with regard to an axis that is parallel to the element’s local \(x\)-\(y\)-\(z\) coordinate system, and \([T]\) is the transformation matrix of DOFs of the element from referencing the element’s shear center axis to referencing the its centroidal axis [20]. Accordingly, the global stiffness matrix of the structural system, and with respect to the single global coordinate system, could be assembled and presented as;

\[
[K]_g = \sum_{\text{NELE}} [K]_e \Gamma \tag{6}
\]

where \([\Gamma]\) is the transformation matrix from the element’s local to the system’s global axis.

### 3.4 Consistent mass matrix

The proper modeling of the sources of mass in a time history analysis can greatly impact the analysis results. Hence, the distributed mass of the member is included by the consistent mass matrix, which is derived based on the Hermite shape function given by Bathe [17]. The mass matrix \([M]_e\) of the element, with length is \(L\), cross-section area \(A\), and uniform mass-density \(\rho\), is expressed as;

\[
[M]_e = \int_L (v^T \rho A v) dx \tag{7}
\]

where \(v\) is the vector of parameters in the shape function.
As a sequence of the above, the global mass matrix \([M]_g\) for the system can be assembled accordingly,

\[
[M]_g = \sum_{i=1}^{NELE} \left( [\Gamma]^T [M]_i [\Gamma] \right)
\]  

(8)

3.5 Rayleigh damping matrix

To reflect the viscous damping within the system, and an in attempt to avoid the complexity in evaluating the distributed damping coefficients along member lengths, the Rayleigh damping model [18] is adopted in the current study. The global damping matrix \([C]_g\) is expressed as

\[
[C]_g = \alpha [M]_g + \beta [K]_g
\]

(9)

where \(\alpha\) and \(\beta\) are proportional coefficients for the system’s mass and stiffness, respectively. They may be calculated based on a first-order elastic modal analysis as

\[
\alpha = \frac{4\pi (\zeta_1 T_1 - \zeta_2 T_2)}{(T_1^2 - T_2^2)}
\]

(10)

\[
\beta = \frac{T_1 T_2 (\zeta_1 T_1 - \zeta_2 T_2)}{\pi (T_1^2 - T_2^2)}
\]

(11)

where, \(T_1\) and \(T_2\) are the natural periods of the first and second modes, respectively; and \(\zeta_1\) and \(\zeta_2\) are the damping ratios for the first and second natural modes, respectively.

4 TIME-HISTORY ANALYSIS PROCEDURE

To perform dynamic analysis of problems in which inertia effects need to be considered, several methods can be adopted, such as direct integration of the system or modal analysis methods [17]. The former should be employed for a nonlinear dynamic response, while the latter is typically used for linear or mildly nonlinear problems. In the direct integration method, the general equation of motion needs to be integrated through the time domain, either implicitly or explicitly. In the implicit method, the global stiffness and mass matrices are formulated and inverted at each time increment to solve a set of nonlinear equilibrium equations. While the explicit technique uses the kinematic state from the previous increment to solve the nonlinear problems at the current state, thereby removing the necessity to invert the system global matrices at each time increment. Accordingly, the explicit method is considered computationally much less expensive than the implicit method.

For the integration of the time steps during the analysis, Newmark’s [16] algorithm is utilized, which is considered unconditionally stable and has been a method extensively applied in research and industry for solving the structural dynamic problems.

The equations of motion in a dynamic time-history analysis are expressed in the incremental form as,

\[
[M]_g \{\Delta u\} + [C]_g \{\Delta \dot{u}\} + [K]_g \{\Delta u\} = \{\Delta F\}
\]

(12)

in which, \(\{\Delta u\}, \{\Delta \dot{u}\}\) and \(\{\Delta u\}\) are the vectors of incremental nodal acceleration, velocity, and displacement, and \(\{\Delta F\}\) is the incremental applied force vector at a time increment. In a seismic analysis, this force is defined in terms of the global mass matrix \([M]_g\) and the increment of the ground acceleration \(\{\Delta \dot{g}\}\), and expressed as,
\[
\{\Delta F\} = -[M]_g [E] \{\Delta \ddot{x}_g\} 
\]

in which \([E]\) is an index vector representing the seismic motion direction(s). Per Newmark’s \(\gamma\) and \(\beta\) factors, the velocity and acceleration at the time \(t+\Delta t\) are calculated as,

\[
\{\ddot{u}_{t+\Delta t}\} = \{\ddot{u}_t\} + (1 + \gamma) \Delta t \{\dot{u}_t\} + \gamma \Delta t \{\ddot{u}_{t+\Delta t}\} 
\]

\[
\{u_{t+\Delta t}\} = \{u_t\} + \Delta t \{\dot{u}_t\} + (0.5 - \beta) \Delta \ddot{t} \{\ddot{u}_t\} + \beta \Delta t^2 \{\dddot{u}_{t+\Delta t}\} 
\]

where, \(\{\dddot{u}_t\}\), \(\{\dot{u}_t\}\), and \(\{u_t\}\) are the acceleration, velocity, and displacement at the previous time \(t\), \(\Delta t\) is a small time-interval. Newmark’s \(\beta\) value is taken as 0.5 to avoid artificial damping, and \(\gamma\) is dependent on the variation of the acceleration within the time interval, which in this research has been assumed as 0.25.

Further, the effective stiffness matrix \([K]_{\text{eff}}\) and the effective external force vector \([\Delta F]_{\text{eff}}\) at time \(t\) are calculated based on the Newmark’s \(N_1\) to \(N_6\) parameters as,

\[
[K]_{\text{eff}} = N_1 [M]_g + N_4 [C]_g + [K]_g 
\]

\[
[\Delta F]_{\text{eff}} = \{\Delta F\} - \left( N_2 [M] + N_5 [C] \right) \{\dot{u}^t\} - \left( N_3 [M] + N_6 [C] \right) \{\ddot{u}^t\} 
\]

with these parameters are given in Table 1. Hence, the incremental equilibrium equation is implemented to solve for the incremental displacement \(\{\Delta u\}\), and accordingly, the increments of velocity, \(\{\Delta \dot{u}\}\), and acceleration, \(\{\Delta \dddot{u}\}\), as follows,

\[
[K]_{\text{eff}} \{\Delta u\} = [\Delta F]_{\text{eff}} 
\]

\[
\{\Delta \dot{u}\} = N_4 \{\Delta \dot{u}\} + N_5 \{\ddot{u}^t\} + N_6 \{\dddot{u}^t\} 
\]

\[
\{\Delta \dddot{u}\} = N_1 \{\Delta \ddot{u}\} + N_2 \{\dddot{u}^t\} + N_3 \{\dddot{u}^t\} 
\]

Table 1. Newmark’s parameters-related factors

<table>
<thead>
<tr>
<th>(N_1)</th>
<th>(N_2)</th>
<th>(N_3)</th>
<th>(N_4)</th>
<th>(N_5)</th>
<th>(N_6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\beta (\Delta t)^2)</td>
<td>(\frac{1}{\beta \Delta t})</td>
<td>(-\frac{1}{2 \beta})</td>
<td>(\gamma)</td>
<td>(-\gamma)</td>
<td>(-\left(\frac{\gamma}{2 \beta} - 1\right))</td>
</tr>
</tbody>
</table>

At the end of each iteration within a time step, the geometry of the model, the displacement \(\{u\}\), velocity \(\{\dot{u}\}\), acceleration \(\{\dddot{u}\}\) and applied forces \(\{F\}\) are updated. Any unbalanced forces and corresponding residual displacement are calculated as,

\[
\{\Delta F^s\} = \left\{\Delta F^s\right\} - \left([M]_g \left\{\Delta \dot{u}\right\} + [C]_g \left\{\Delta \ddot{u}\right\} + \left\{\Delta R\right\}\right) 
\]

\[
\{\Delta u\} = [K]_{\text{eff}}^{-1} \{\Delta F^s\} 
\]

Iteration continues until the below convergence criteria are satisfied.

\[
\{\Delta \dot{u}\}^T \{\Delta \dot{u}\} < \text{TOL} \ast \{\Delta \dot{u}\}^T \{\Delta \dot{u}\} 
\]

\[
\{\Delta F\}^T \{\Delta F\} < \text{TOL} \ast \{\Delta F\}^T \{\Delta F\} 
\]

in which, TOL is the required tolerance on the numerical iteration. In general, the model geometry is updated using the residual displacement, and the procedure is repeated until the convergence criteria is satisfied.
5 NUMERICAL IMPLEMENTATIONS

The derived element formulations and the proposed numerical procedure have been implemented into the educational analysis software MASTAN2-v5.0 [19]. A new MSA_Seismic module provides access to defining input ground motions and includes a database of many earthquake records. Consequently, the incremental numerical procedure for first-order and second-order elastic time-history analyses is presented within the dynamic time-history analysis module.

6 VERIFICATION EXAMPLES

In this section, three comprehensive examples are presented to illustrate the application of the proposed line element for a more accurate and robust dynamic analysis.

6.1 Example 1 – Second-order time-history analysis of cantilever beam under dynamic excitation

This example illustrates the dynamic response of cantilever beams under different sources of excitations. The nonsymmetrical channel-section cantilever beam shown in Fig. 4 has a length of 5.0 m, with dimensions of h = 300 mm, b₁ = 200 mm, b₂=100, flange plate-thickness tᵣ = 16 mm, and web thickness tₜ = 10 mm. Young's modulus is E = 210 GPa, and Poisson's ratio is ν = 0.3. The member self-weight is not considered in the dynamic analysis, and a concentrated force (P) of 8 kN is used to back-calculate a mass (M=P/g) at the free end of the beam. In this example, a sophisticated shell finite-element model (SFEM) is utilized to provide benchmark results for the aforementioned effects and complexities.

Concerning modal frequency analysis, the first three natural periods obtained from the SFEM are provided in Table 2 for comparison.

To demonstrate time history capabilities, the cantilever beam is subjected to three different vertical support excitations, including simple, sine, and saw-tooth functions, as shown in Fig. 5. Note that the simple excitation, expressed in Equation (25), is only applied for the first full second, while the total step time (i.e., 10 seconds) is used to examine the freely vibrated behaviors after the initial excitation is discontinued. The time increment for the analysis is 0.02s, and the damping ratio is assumed to be 5%. The Newmark’s coefficients γ and β are taken as 0.5 and 0.25, respectively. Accordingly, the dynamic responses (vertical displacement vs. time curves) of the cantilever beam under the three types of excitations are shown in Figs. 5(a) to (c). It can be observed that the proposed method employing the line element (LFEM) that accounts for the non-symmetry of the section is matching results with the sophisticated SFEM.

\[ \frac{a(t)}{g} = \begin{cases} 
0.2t & t \leq 1 \\
0.0 & 1 \leq t \leq 10 
\end{cases} \] 

(25)
(a) Cantilever beam model and section dimension  (b) ABAQUS shell finite-element model (SFEM)

Figure 4. Cantilever beam

Table 2. Comparison of natural vibration periods

<table>
<thead>
<tr>
<th>Mode Description</th>
<th>SFEM</th>
<th>Present study</th>
<th>Diff. %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st Mode (F-T)</td>
<td>0.5744</td>
<td>0.5721</td>
<td>-0.40%</td>
</tr>
<tr>
<td>2nd Mode (F-T)</td>
<td>0.2977</td>
<td>0.2944</td>
<td>-1.11%</td>
</tr>
<tr>
<td>3rd Mode (Axial)</td>
<td>0.0101</td>
<td>0.0101</td>
<td>0.00%</td>
</tr>
</tbody>
</table>

Note: F-T, and Axial refer to flexure-torsion, and axial natural modes, respectively.

(a) simple excitation  (b) sine function  (c) triangular function

Fig. 5 Different transient functions applied for the cantilever beam

Figure 6 Dynamic response of cantilever beam under dynamic excitations

6.2 Example 2 – An individual column under seismic excitations

This example presents the time-history response of a channel section steel column with a lumped mass at its top under the seismic excitations. The section is C250x30, and the dimensions are shown in Figure 7. The column length is 2000mm, and the Young’s modulus and the poison’s ratio are 210GPa and 0.3, respectively. The boundary conditions of this channel column are fixed at one end and free at the other, in which a concentrated force of 15kN is applied as an additional mass. The model is shown schematically in Figure 11. This example focuses on generating the dynamic responses of the channel column under the 1940 EI Centro and the 1971 San Fernando ground excitations (Figure 8) using different line elements, i.e., the nonsymmetric warping 7 DOFs, the conventional symmetric 7 DOFs and the 6DOFs line elements. The time vs. displacements and the time vs. accelerations of the column under two seismic excitations are given in Figure 9.
Figure 7. Boundary conditions of individual column

Figure 8. Seismic excitations

Figure 9. Dynamic response analysis results of the individual columns

6.3 Example 3 – An upright frame under seismic excitations

In this example, the second-order dynamic time history analysis is performed for the steel rack uprights frame. The geometry of the upright frame is shown in Figure 10, with 1000kg additional mass placed at the top of each column. The Young’s modulus and Poisson’s ratio is
210Gpa and 0.3, respectively. The reduced thickness method is used to consider the perforations on the upright columns. The cross-sectional dimensions of the columns and the diagonal brace are plotted in Figure 11. The boundary conditions at the base of the upright are fixed. The seismic excitation direction is perpendicular to the column and in the plane of the upright frame. The time acceleration curves of the earthquake are taken from the 1940 El Centro and the 1971 San Fernando earthquakes. In this model, the diagonal braces are semi-rigidly connected to the column, details of which can be found in [25].

Figure 10. Frame geometry

(a) Frame overview  (b) Dimensions of frame  (c) Column and brace section

Figure 11. Dynamic response analysis results of the upright frame

7 CONCLUSIONS

In this research, an efficient numerical framework, including line finite element method (LFEM) formulations and direct time-integration method using Newmark’s approach is
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proposed for modeling the dynamic responses of thin-walled steel members of arbitrary cross-sectional shapes. As demonstrated in the above examples, it can be important to consider the Wagner effect in the analysis members with nonsymmetrical cross-sections – this is because of twist deformations are likely induced under dynamic excitations. In contrast, a 6- or 7-DOF conventional LFEM using the Hermite beam-column element is unable to capture such behavior, and hence, may not provide an accurate estimation on the dynamic response. From the findings reported in this paper, the following conclusions can be drawn:

- The proposed second-order elastic time-history analysis method, which accounts for nonsymmetrical sections, appears to be accurate and can be used in the practical design of metal structures. It is further believed that the dynamic response of systems of symmetrical and nonsymmetrical section members may be significantly different when subject to complex earthquakes.
- From the examples presented herein, it can be concluded that the conventional LFEM with symmetrical section assumption, without consideration of the potential for the noncoincidence of the shear center and the centroid, may not be able to predict the dynamic behavior of nonsymmetrical section members, and therefore should be used cautiously in designing structural systems with thin-walled members.
- This research may be helpful for the efficient analysis of systems of the thin-walled members, especially for the use in intense seismic regions.

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.

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