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The GCM-FE Method for the advanced analysis of steel frameworks and connections

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Abstract
The Generalised Component Method (GCM) has recently been formulated for analysing the nonlinear response of connections, incorporating the elastic and inelastic stiffnesses, the post-ultimate softening as well as the ductility of each component, thus defining a multilinear force-displacement curve for each component. By assembling the components of a connection in a model comprising multi-linear springs, as per the Component Model concept, the full-range nonlinear connection response can be obtained including the ultimate capacity, pre- and post-ultimate load-deformation responses, and the ductility before fracture and complete failure. In this paper, the GCM is implemented in an FE analysis framework of steel structural frames. The paper explains the key elements of the FE modelling, such as the integration of the connection model with adjoining members and the use of connector elements to represent components by imposing kinematic constraints and specifying nonlinear force versus displacement behaviour. The FE implementation is briefly described including the use of published Python scripts and Matlab code. The framework, termed the GCM-FE joint modelling method, is then first validated against experimental results for the bolted moment end-plate connection. Subsequently, the GCM-FE method is used to analyse a two-storey four-bay irregular steel frame and shown to provide substantially greater insight into the performance of connections than the simplified joint models used in traditional analysis methods. Specifically, the GCM-FE analysis not only provides the ultimate resistance and failure mode of the frame, considering both members and connections, but also accurately predicts the load-redistribution process inside the connections and the resultant effect on the structural framework. The GCM-FE method presents a major step towards the complete direct design of steel frameworks by advanced analysis, which includes both members and connections. This design-by-analysis approach lays the foundation for the next generation of structural design standards in which the strength and structural safety checks are performed in a single step at system level.

1. Introduction

Joints feature complex geometries and are difficult to model in finite element analysis. At present, it is not a practical proposition to discretise each component of joints in routine design

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but rather, models that capture the stiffness and strength at member action level are more common, typically moment-rotation response models. Several types of models can be used to determine the mechanical behaviour of joints, e.g. [1] provides an overview of available analytical, empirical, experimental and mechanical models. Component-based mechanical models have been developed by several researchers for the prediction of moment-rotation curves for a wide range of connections, where the number of physical governing parameters is limited [2].

The Component Method divides a connection into a system of springs, each representing a component of the connection transferring actions. Early versions of the Component Method [3, 4] were developed into the format that is now implemented in EN 1993-1-8 [5], and allows the initial semi-rigid stiffness and flexural resistance of joints to be calculated. Bilinear spring models were later presented for modelling both the initial elastic response of a spring and the subsequent inelastic response [6, 7]. Recent research at the University of Sydney has extended the use of the Component Method to the ultimate state and the post-ultimate range of steel joints, where the ultimate state can be due to either fracture of components in the tension zone or buckling in the compression zone [8]. The proposed model has been confirmed as an accurate tool for studying the full-range behaviour of steel connections. The moment versus rotation responses of steel connections can also be predicted by component based finite element method, in which the inelastic behaviour of steel plates is allowed for through material nonlinearity, and the behaviour of components, e.g. bolts, and welds, is treated by introducing nonlinear springs [9-10].

This paper summarises a finite-element analysis framework that directly allows for the full-range behaviour of steel beam-to-column joints in the analysis of steel frames, by incorporating Generalised Component Method-based joint models in the numerical model. All components in each connection are explicitly modelled in the analysis and are assigned with full-range constitutive models. This new analysis methodology is termed the Generalised Component Method-based finite element (GCM-FE) analysis of steel frames. Full details of the GCM-FE are provided in [11]. The GCM-FE analysis approach enables the complete single-step design-by-analysis approach to be performed, in which the effect of axial forces in beams and the semi-rigidity and failure of connection are explicitly considered. The direct GCM-FE approach guarantees more uniform structural system reliability, and provides a greater understanding of the behaviour of the structural system and its mode of failure.

2. GCM-FE analysis approach

2.1 Framework of GCM-FE analysis

The concept of employing GCM-FE analysis is illustrated in Fig. 1. For the steel frame to be analysed (Fig. 1a), each beam-to-column joint is modelled using a series of rigid bars and extensional springs, distinguishing the separate sources of deformability (Fig. 1b). The vertical rigid bar in the middle is aligned with the column centreline within the connection region, and is rigidly connected to the upper and lower columns at the top and bottom ends, respectively, with all the translational and rotational degrees of freedom constrained to ensure continuity in displacements and slopes. Each side rigid bar represents the end of the respective adjacent beam, and is rigidly connected to the beam at the intersecting node. The axial springs aligned with the beam flanges and bolt rows represent the deformability of the connection components.

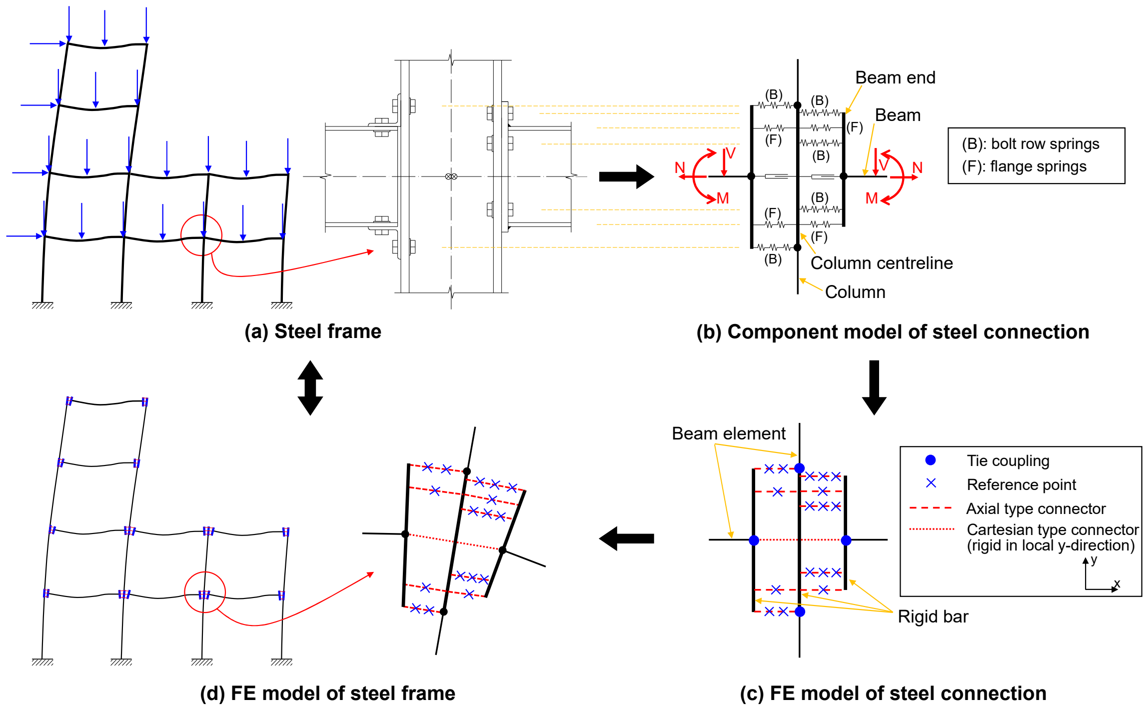

Figure 1: Methodology of Component-Method based analysis of steel frames

To implement the GCM-FE analysis, FE software such as Abaqus or OpenSees is employed to develop the FE model, as shown in Fig. 1c. The rigid bars in the joint model can be represented by rigid elements, very stiff elastic beam-column elements which are considered as equivalent rigid elements, or rigid element type constraints. The axial springs can be represented by spring elements, connector elements, truss or link elements with appropriate axial stiffness, or link type constraints. The choice of modelling methods for the rigid bars and axial springs depends on their availability in the FE software, chosen as Abaqus in this study.

Having developed the GCM-FE model for a steel frame, the behaviour of the frame can be obtained in a computational cost-effective fashion and also with high accuracy, especially when semi-rigid connections are employed and their effect should be allowed for, as shown in Fig. 1d. The GCM-FE analysis approach has the following apparent advantages. Firstly, the full-range constitutive model is proposed (Section 2.2) for each component contributing to the deformability of the steel joint, simulating the full-range behaviour of components including gradual plasticity and component failure. The full-range component models, combined with the plasticity models defined for the structural member materials, enable the full-range behaviour of the steel frame including the ultimate resistance and failure mode to be obtained in a single run of FE analysis. Secondly, in the frame analysis the forces in all components are simultaneously determined, which through the full-range constitutive model of each component in a connection and the assembly of the component responses explicitly reflect the effect of axial force applied to the connection on its flexural behaviour, thus circumventing the need of introducing amended moment-rotation models for connections subject to the effect of axial forces. Thirdly, the full-range constitutive models proposed for the components include the behaviour of each component under both tension and compression, in contrast to the conventional component models which define the behaviour of a component under either tension or compression. This allows the GCM-

FE analysis to be performed for loading conditions which generate reversed bending moments or catenary forces at the connections. Fourthly, through the two new connection components incorporated in the GCM-FE analysis framework, i.e. the Bolt slip component and the Beam flange contact component, the GCM-FE analysis approach is capable of capturing the complicated behaviour of bolted connections under large joint rotation, as well as the resultant change of load transfer path through the connection and the effect thereof on the frame response.

2.2 Constitutive models for components

The positive-to-negative full-range constitutive relation of each component type is a key element of the GCM-FE framework. Fig. 2 summarises the constitutive relations of all the component types in a steel beam-to-column connection, including the two new components proposed in this study, i.e. the Bolt slip component and Flange gap component (the last row in Fig. 2), as well as the components specified in EN 1993-1-8 [5]. The Web cleat in bending component (i.e. Component 21) is developed in [12]. All the components are categorised into 11 groups according to their mechanical behaviour.

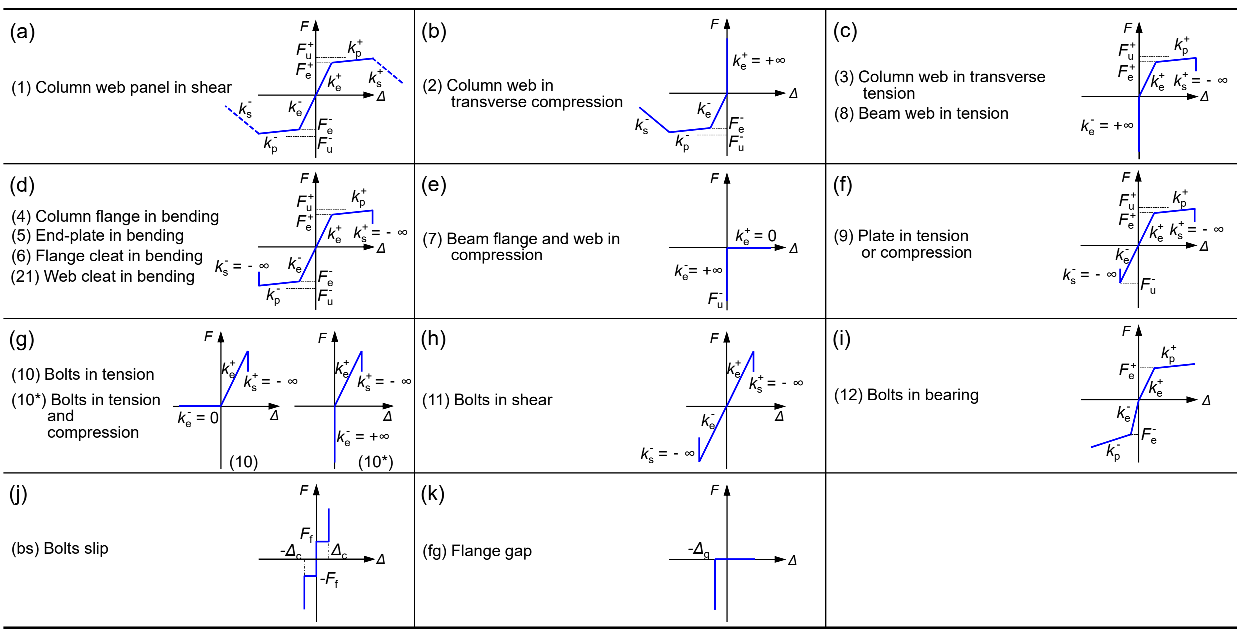

Figure 2: Component properties

Full details of the parameters defining the elastic, inelastic and post-ultimate properties of each of the component groups shown in Fig. 2 are given in [11]. For most groups, the parameters describing the elastic range can be obtained from EN 1993-1-8, whereas other parameters defining the inelastic behaviour and ductility are obtained from the literature, where available, or simple assumptions are made, e.g. that the post-ultimate range is negligible (marked as $k_s = \pm\infty$), such as in the case of Component 10 Bolts in tension. Full-range models covering the pre- and post-ultimate inelastic stiffnesses have not yet been developed for some components. However, appropriate conservative values can be assigned to these stiffnesses based on engineering judgement until component models become available.

It is noted that while the constitutive models shown in Fig. 2 include the most fundamental properties of the components, the models are not unique. Rather, any model deemed reasonable and appropriate can be used to define the constitutive relations. Also, the GCM-FE analysis allows the ductility of a connection to be determined provided the ductility of each component is known. This is a major advantage of the GCM-FE analysis as it can be employed to ensure the structural integrity of connections is not compromised in the range of analysis.

3. Implementation of GCM-FE analysis in Abaqus

3.1 Modelling of joints and members

In developing the GCM-FE model in Abaqus, the rigid bars are represented by very stiff elastic beam elements with an area of 1.0×108 mm2 and a moment of inertia equal to 1.0×1012 mm4 in order to give them high axial and flexural stiffnesses, respectively. Stiff beam elements instead of analytical or discrete rigid beam elements are used because the former are available in both Abaqus/Standard (the implicit solver in Abaqus) and Abaqus/Explicit (the explicit solver in Abaqus), while the latter are only available in Abaqus/Standard.

The behaviour of each component is represented by an Axial-type connector element. Compared to the spring element and multi-point constraint, the connector element (abbreviated as "connector" herein) embedded in Abaqus is more powerful, as it provides an easy and versatile way to model many types of physical mechanisms whose geometry is discrete (i.e., node-to-node), yet the kinematic and kinetic relationships describing the connection are complex [13]. The many types of connectors can impose kinematic constraints, and include (nonlinear) force versus displacement (or velocity) behaviour in their unconstrained relative motion components with plastic behaviour, stopping and locking mechanism, as well as failure conditions defined.

Corresponding to a bolt row or a beam flange there are normally multiple connectors (i.e. components). Hence, for such bolt row or beam flange, as shown in Fig. 1c, a series of reference points, the number of which is equal to the number of connectors minus one, are created first between the rigid bars. The connectors are then created to connect each rigid bar to its adjacent reference point, representing the component next to the rigid bar, or to connect two adjacent reference points, representing internal components. For each Axial-connector, the full-range behaviour of the component it represents is defined as a nonlinear force versus displacement curve.

The Axial-connectors transfer the bending moment at the beam end, through a force couple, and the axial force in the beam to the column. In addition to the bending moment and axial force, the shear force in the beam has to be transferred to the column. Ideally, at each bolt row, a Cartesian-type connector with rigid behaviour in the vertical direction would be defined between the rigid bars, simulating the transfer of the shear force through the bolts, as shown in Fig. 3a. However, under large rotations of the connection, the rigid bar representing the beam end rotates significantly, and all the Cartesian-type connectors cannot remain horizontal at the same time, causing numerical problems. Therefore, only one Cartesian-type connector is used. The Cartesian-type connector can be located at any bolt row, or aligned with the beam centreline, without leading to notable differences. Herein, the Cartesian-type connector aligns with the beam centreline, as shown in Fig. 3b and Fig. 1c.

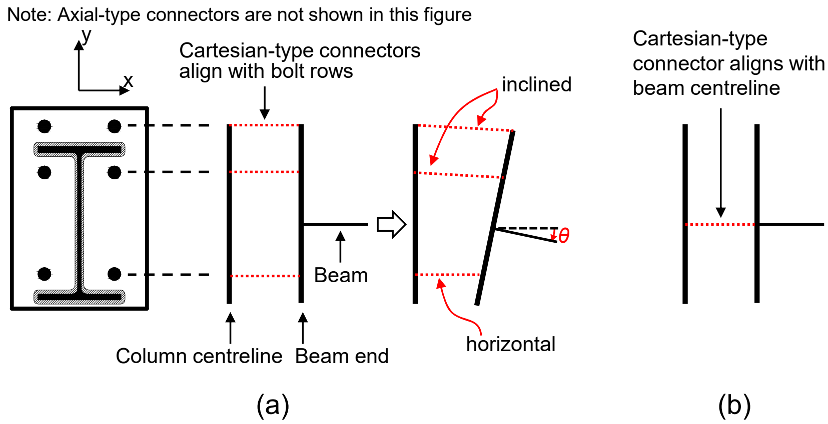

Figure 3: Cartesian-type connector to transfer the shear force

The beam is modelled using Beam elements (B31 element type in Abaqus), and connects to the rigid bar representing the beam end at the intersecting node through a Tie constraint, constraining all translational and rotational degrees of freedom of the rigid bar to those of the node at the beam end. Similarly, the column is also modelled using B31 elements, and connects to the rigid bar representing the column centreline at the end of the rigid bar through a Tie constraint.

3.2 Automatic modelling using Python scripts

Each connection has many components, assembled in several rows aligned with the bolt rows and beam flanges. The calculation of coordinates for all the components in every connection in the global coordinate system of the steel frame is labour-intensive and error-prone if done manually, especially for high-rise buildings that have a great number of connections with a variety of connection configurations, let alone manually creating the connectors (i.e. components) in finite-element software. This can be a prohibitive factor for the GCM-FE analysis to be widely adopted in the practical analysis and design of steel frame buildings.

To overcome this limitation and thereby promote the wide use of GCM-FE analysis, the GCM-FE modelling is automated in Abaqus through Python programming. The Python script calculates the coordinates of all nodes for both the structural members and connectors in the connections, then creates the GCM-FE model and writes this to an input file in a format suitable for submission to Abaqus. The script is provided in Appendix C of [14], enabling fast, convenient and automatic creation of GCM-FE analysis models requiring only the input of the basic information of the steel frame, as described in detail in [11, 14].

4. Validation of GCM-FE joint model

This section demonstrates the application and accuracy of the GCM-FE method when applied to the bolted moment end-plate connection. The joint behaviours under pure bending as well as combined bending and axial force are examined. Similar validation of the method for the top-and-seat angle connection and the web angle connection are provided in [11].

The full-range behaviour of extended end-plate connections was evaluated in [15]. Three connection configurations were examined, which had identical geometric and material properties except that the first (EP10 connection) had a 10 mm thick end-plate, the second (EP10BP connection) had a 10 mm thick end-plate and an additional backing plate, and the third (EP20BP connection) had a 20 mm thick end-plate and a backing plate. Fig. 4a shows the geometry of the connections. The EP10BP connection was subjected to bending moment, while the EP10 and EP20BP connections were tested under two distinct loading conditions, namely, bending and combined bending and axial force.

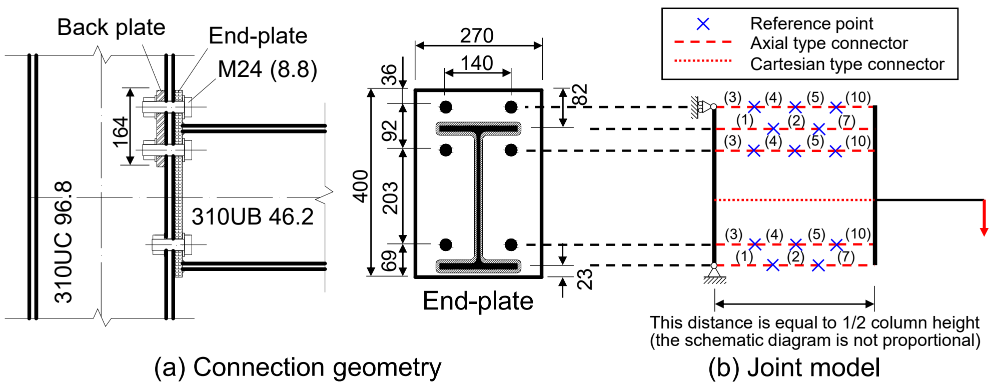

Figure 4: Joint model for the end-plate connections in [15]

GCM-FE joint models are developed for the three connection configurations, as shown in Fig. 4b. The three joint models are identical in terms of active components. The components corresponding to the bolt rows are (3) Column web in tension, (4) Column flange in bending (the backing plate is considered as a part of the column flange), (5) End-plate in bending, and (10) Bolts in tension, while the components corresponding to the beam flanges are (1) Column web panel in shear, (2) Column web in compression, and (7) Beam flange and web in compression. The characteristics of the constitutive models for each component are as described in [11], and the model parameters are adopted as those calculated and defined in [8], in which the Generalised Component Method capable of predicting the full-range moment-rotation response of steel joints was proposed, and are validated against experimental results in [15]. The parameters of the constitutive model for each component considered in the GCM-FE joint models are available in [11].

As a result of the relatively thin end plate, the EP10 and EP10BP connections failed in an end-plate bending failure mode, which featured significant plastic deformation of the end-plate in the tension zone and subsequent fracture initiation and propagation in the end-plate along the weld. The above failure process was reflected in the experimental moment-rotation curves as several abrupt decreases in resistance in the post-ultimate range, as shown in Figs. 5 and 6. In Fig. 5, the “B” curves correspond to the connection subjected to pure bending while the “B+T” curves correspond to the connection subjected to combined bending and axial tensile force. It was observed that the additional axial tensile force accelerated the failure of the end-plate in the tension zone, leading to smaller resistance and inferior joint rotation capacity. The EP20BP connections had a 20 mm thick end plate which was thick enough to prevent it from failing in bending. As a result, the connections failed due to the buckling of the unstiffened column web in the compression zone, which became the critical component of the connection. The experimental moment-rotation curves of the EP20BP connections are shown in Fig. 7, in which the “B” curves correspond to the connection subjected to pure bending while the “B+C” curves correspond to the connection subjected to combined bending and axial compressive force. The additional compressive force induced earlier buckling of the column web.

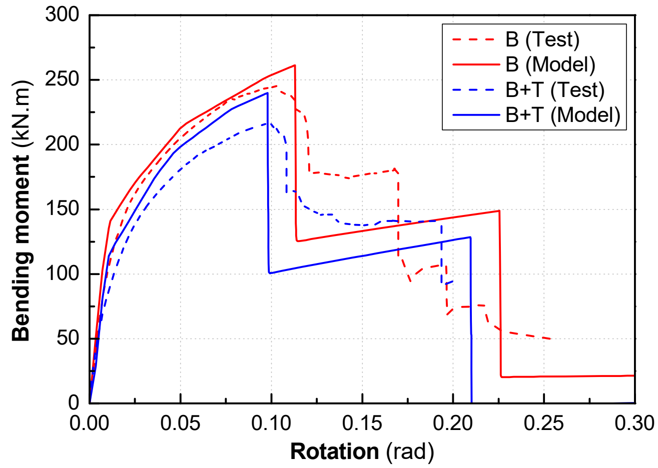

Figure 5: Model prediction for end-plate connection EP10 in [15]

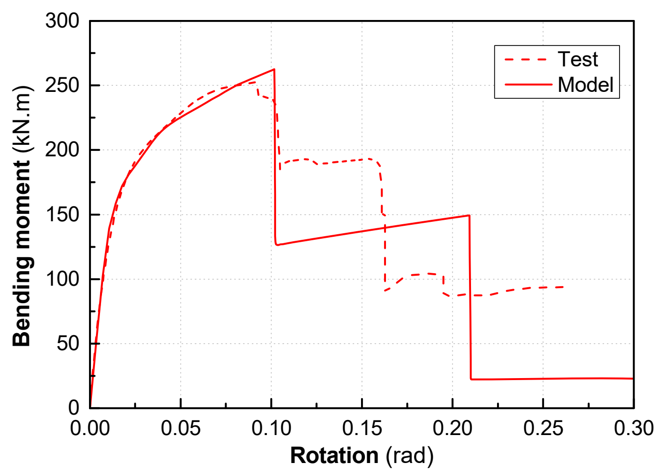

Figure 6: Model prediction for end-plate connection EP10BP in [15]

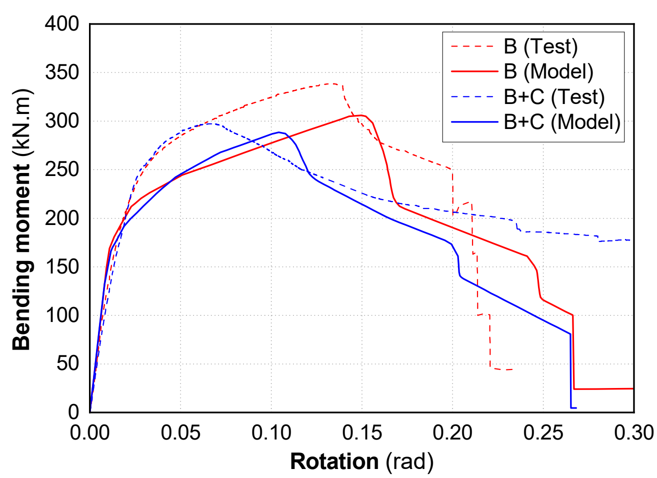

Figure 7: Model prediction for end-plate connection EP20BP in [15]

Figs. 5, 6 and 7 show the predicted and experimental moment-rotation curves of the EP10, EP10BP and EP20BP connections, respectively. The GCM-FE joint models give reasonably accurate predictions of the flexural behaviour of all three connection configurations, agreeing well with the experimental results in terms of the initial stiffness, ultimate resistance and rotational ductility. Especially, the joint models capture the connection failure modes, i.e. end-plate bending failure at the first bolt row in the EP10 and EP10BP connections and buckling of column web due to compression in the EP20BP connection, and the post-ultimate responses of the connections. Moreover, the joint models are also capable of reflecting the effect of the axial force on the joint flexural behaviour.

5. Frame analysis example

Having validated the GCM-FE joint model for the bolted moment end-plate connection, a GCM-FE analysis is performed on a two-storey four-bay irregular frame, which is adopted from [16]. Fig. 8 shows the frame layout, loading conditions and member profiles. European sections (i.e. IPE and HEB sections) are used for the beams and columns, and the material is steel S275, with a modulus of elasticity of 210,000 MPa and a yield stress of 275 MPa.

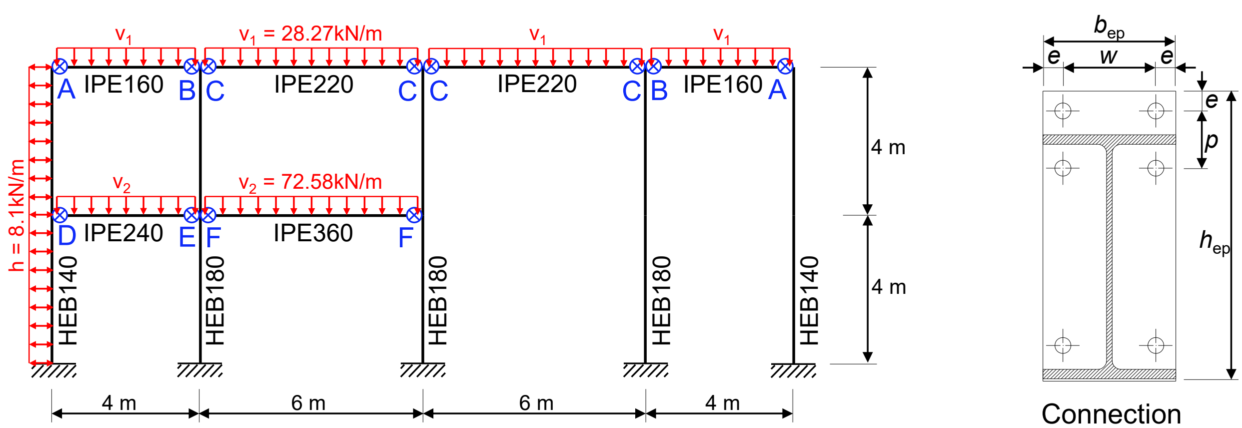

Figure 8: Example: two-storey, four-bay irregular frame (redrawn from [16])

Bolted moment end-plate connections are used at all beam-to-column connections. There are six connection configurations, the geometric parameters of which are given in Table 1. The flexural behaviour of each connection configuration can be evaluated from the GCM-FE joint model presented in Section 4. In developing the joint models, the model parameters for the connection components are defined as shown in Table 2. The initial stiffness and elastic limit are determined from EN 1993-1-8 [5]. The inelastic stiffnesses (k_p^-, k_p^+) of the components in the compression zone (i.e. Components 1 and 2), the tensile component in the tension zone (i.e. Component 3), and the bending components in the tension zone (Components 5 and 6) are assumed as 5%, 2% and 5% of the initial stiffness (k_e^-, k_e^+) of the respective components. The ultimate resistances (F_u^-, F_u^+) of the components in the compression zone, the tensile component in the tension zone, and the bending components in the tension zone are assumed as 1.4, 1.2 and 1.5 times the elastic limit (F_e^-, F_e^+) of the respective components. The softening stiffnesses (k_s^-, k_s^+) of the components in the compression zone are assumed as –1% of the initial stiffness (k_e^-, k_e^+) of the components. The above assumptions are adopted for simplicity, and the rationale behind the assumptions is explained in [11]. However, evidently, any other constitutive model deemed appropriate can be used for the components in the GCM-FE analysis. Table 1 summarises the initial stiffnesses, ultimate resistances and failure modes of each connection, as obtained using the GCM-FE analysis.

Table 1: Properties of connections

Joint	Beam	Column	Bolt	h_{ep} (mm)	b_{ep} (mm)	t_{ep} (mm)	p (mm)	w (mm)	e (mm)	Init. Stiff. (kN.m/rad)	M_u (kN.m)	Failure mode
A	IPE160	HEB140	T16	245	140	10	90	80	30	5,900	31.8	epb
B	IPE160	HEB180	T20	240	140	12	80	80	30	8,820	39.2	cws/epb
C	IPE220	HEB180	T20	295	140	14	70	80	30	15,300	68.4	cwt
D	IPE220	HEB140	T22	295	140	16	70	80	30	11,900	51.8	cws/cwt
E	IPE220	HEB180	T22	295	140	18	70	80	30	15,800	68.6	cwt
F	IPE360	HEB180	T22	435	140	16	70	80	30	39,400	114	cwt

Table 2: Coefficients used for the constitutive models of components

Component	k_e^+	F_e^+	k_p^+	F_u^+	k_s^+	k_e^-	F_e^-	k_p^-	F_u^-	k_s^-
(1) Column web in shear	k_e^-	$-F_e^-$	k_p^-	$-F_u^-$	k_s^-	EC3	EC3	$0.05k_e^-$	$1.4F_e^-$	$-0.01k_e^-$
(2) Column web in compression	$+\infty$	–	–	–	–	EC3	EC3	$0.05k_e^-$	$1.4F_e^-$	$-0.01k_e^-$
(3) Column web in Tension	EC3	EC3	$0.02k_e^+$	$1.2k_e^+$	$-\infty$	$+\infty$	–	–	–	–
(4) Column flange in bending	EC3	EC3	$0.05k_e^+$	$1.5F_e^+$	$-\infty$	k_e^+	$-F_e^+$	k_p^+	$-F_u^+$	$-\infty$
(5) End-plate in bending	EC3	EC3	$0.05k_e^+$	$1.5F_e^+$	$-\infty$	k_e^+	$-F_e^+$	k_p^+	$-F_u^+$	$-\infty$
(7) Beam flange & web in comp.	0	–	–	–	–	$+\infty$	–	–	–	–
(10) Bolt in tension	EC3	–	–	–	$-\infty$	0	–	–	–	$-\infty$

The vertical and horizontal loads are applied proportionally on the steel frame up to and beyond the design loads shown in Fig. 8 until the complete failure of the frame. As shown in Fig. 9, the frame failed at the first beam level of the left-most bay (referred to as Beam-1-1) at a load ratio of 1.031, where the load ratio is defined as the ratio of the applied load to design load. The failure initiated at the right-hand-side connection of Beam-1-1, referred to as Conn-R. The failure of Conn-R is characterised by the progressive failure of the tensile bolt rows, from top to bottom, as shown in Fig. 9b which shows the force transferred through the bolt rows and bottom flange of Conn-R. Fig. 9c shows the bending moments in Beam-1-1 and its right and left end connections, Conn-R and Conn-L. It is observed that the failure of Conn-R results in significant reduction in the flexural resistance of the connection, which in turn leads to increases in the bending moments in both Beam-1-1 and Conn-L, triggering the development of a plastic hinge near the centre of Beam-1-1 and the failure of Conn-L. The failure of Conn-L marks the complete failure of the steel frame.

The observed load redistribution process demonstrates that the incorporation of the component-based connection model in the GCM-FE analysis allows the response of components inside the connections and the resultant effect on the entire structure to be captured. Moreover, Fig. 9c shows that Conn-R failed at the bending moment of 70.5 kN·m, which is higher than the ultimate flexural resistance (i.e. 68.4 kN·m) shown in Table 1. This is because the applied horizontal load induces a compressive force of 25.7 kN in Beam-1-1, postponing the failure in the tension zone of Conn-R and thereby increasing the ultimate flexural resistance of the connection. The GCM-FE analysis accurately and explicitly allows for the effect of axial force on the flexural response of the connection, which in turn affects the response of the entire frame. This capability, and the understanding of the connection behaviour it provides, present a major advantage of the GCM-FE analysis over conventional modelling of connections at stress resultant level, typically using moment-rotation relations.

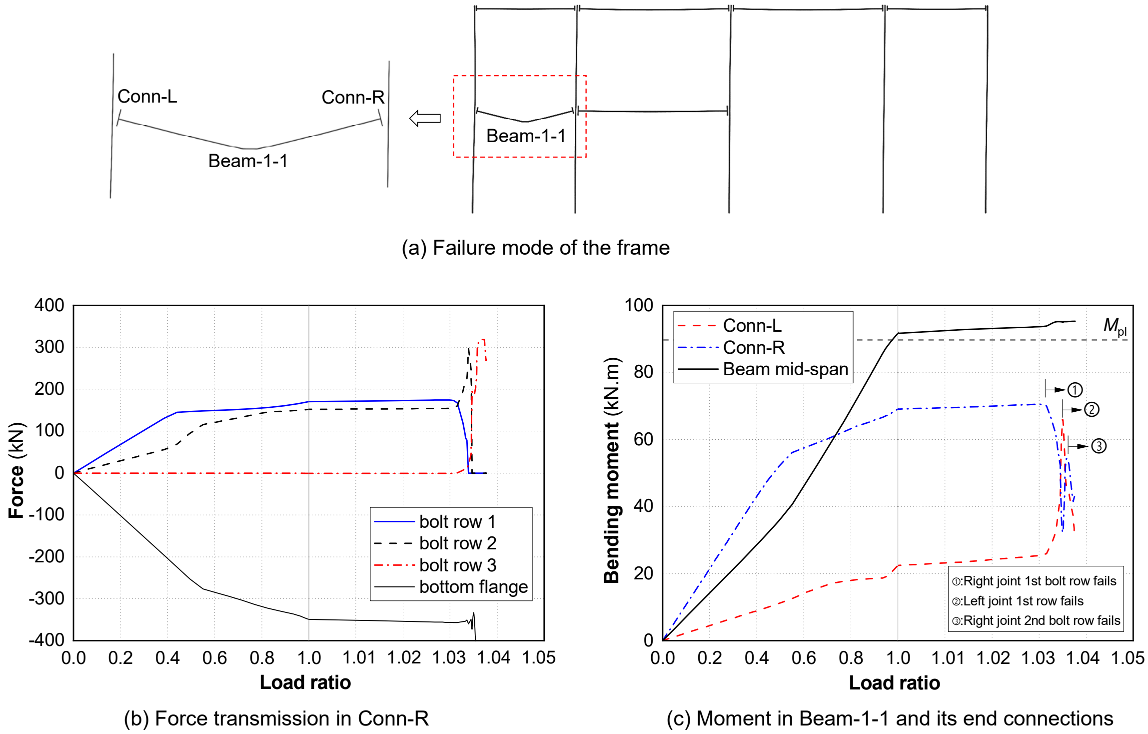

Figure 9: Example: two-storey, four-bay irregular frame (redrawn from [16])

6. Conclusions

This paper presents a new analysis approach that incorporates the FE macro-elements-based Component Method joint model in the FE modelling of steel frame buildings. The new analysis approach is termed Generalised Component Method-based finite element (GCM-FE) analysis. The most fundamental aspects and principles of the GCM-FE analysis approach are established in the paper, including the framework of GCM-FE analysis, the constitutive models for the connection components and the implementation of GCM-FE analysis in commercial numerical software.

The GCM-FE joint modelling is the core of the GCM-FE analysis approach, and in this paper, is used to produce the full-range flexural behaviour of the bolted moment end-plate connection, indicating good agreements with experimental results in the literature. Moreover, a GCM-FE analysis is performed on a two-storey four-bay irregular steel frame. The analysis not only obtains the ultimate resistance and failure mode of the frame, but also captures the load-redistribution process inside the connections and the resultant effect on the entire structure.

The GCM-FE analysis framework and the corresponding implementation method proposed in this study enable the complete single-step design-by-analysis approach to be performed, which is inherently more accurate and faster than the current two-step member-based design approach. It guarantees more uniform structural system reliability, and provides a greater understanding of the behaviour of the structural system and its mode of failure. Accounting for failure of both members and connections, the GCM-FE analysis approach will pave the way for introducing computer-based direct design of steel structures in the structural engineering community. Moreover, intended to explain and verify the GCM-FE analysis framework, the example and connection models in this paper are all 2D. It should be noticed that the GCM-FE framework and models are readily extended to 3D.

Acknowledgments

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