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Analysis of pallet rack beam members through a nonlinear GBT formulation with sectional constraints

Jordi Bonada, Miquel Casafont, Francesc Roure, Mª Magdalena Pastor

Correspondence

Dr. Jordi Bonada
Universitat Politècnica de Catalunya
Escola Tècnica Superior d'Enginyeria Industrial de Barcelona
Av. Diagonal 647
08028 Barcelona
Email: jordi.bonada@upc.edu

Abstract

Thin-walled open cross-sections are commonly used in steel storage pallet rack structures. In some cases, open sections are employed as beam members. These beam members usually have weld spots along their length in order to reduce the distortional deformation under a flexural load. On the other hand, beam members are typically connected to columns by means of claws, which typically exhibit a nonlinear moment - rotation relationship. These two characteristic should be taken into account in order to properly reproduce their structural response.

In this paper, the Generalised Beam Theory (GBT) is used to perform a geometrical nonlinear analysis of a pallet rack beam members taking weld spots and the beam-to-column connection behavior into account. The influence of the number and location of weld spots, as well as the beam-to-column connection behavior, in the distortional deformation of a beam member is evaluated by means of GBT nonlinear analyses. The results show that the use of a small number of weld spots could be enough to minimize the beam distortional deformation. Moreover, the stiffness of the beam-to-column connection also has influence on the beam distortional deformation. Finally, GBT results are successfully compared with shell finite element analyses.

Keywords

Generalised Beam Theory (GBT), sectional constraints, thin-walled members, FEA, nonlinear analysis

1 Introduction

The basic structural members of storage pallet racking systems are columns, beams and bracings or diagonals. The storage loads are applied to the beams, which transmit the forces to the columns. Open thin-walled cross-sections can be used as beam members in pallet rack structures. Nevertheless, these beam members usually have weld spots along their length in order to reduce the distortional deformation under a flexural load.

The beam-to-column connections in most pallet rack frames are made through claws or buttons. This kind of joints usually have a nonlinear behaviour between the flexural moment and the rotation angle [1-3]. A proper reproduction of these connections is necessary in order to obtain a right internal force diagram and displacement field when performing a structural analysis.

The finite element method has been used to analyse the structural behaviour of pallet rack structures. Traditional beam elements are commonly used for global analyses of whole pallet rack structures because of their low computational cost. However, these models cannot reproduce sectional deformations, sectional constraints as weld spots, perforations or local effects. Conversely, shell finite element models can handle sectional deformations, perforations, sectional constraints and more particularities of pallet rack structures [4-7]. Nevertheless, a shell finite element model of a whole pallet rack structure could present a high computational cost. Furthermore, the structural internal force diagram cannot be directly obtained with a shell finite element analysis.

The use of advanced beam models with a lower computational cost, such as Generalised Beam Theory (GBT), can be one alternative to analyse pallet rack structures considering some of their particularities. In fact, GBT has been extended to orthotropic materials [8, 9], different kind of cross-sections [10, 11], perforated members [12, 13], geometric and material nonlinearities [14-19], etc. Furthermore, it is possible to introduce arbitrary support conditions, pure modal springs, sectional constraint equations, etc. [20-21] which can be applied to reproduce the connection characteristics between different structural members in a pallet rack frames.

In this paper, the influence of weld spots and the beam-to-column connection behaviour is analysed by means of a GBT geometrical

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nonlinear analysis. First of all, sectional constraint equations are introduced into GBT formulation to reproduce weld spots. Moreover, a nonlinear pure modal spring is defined into the GBT model to reproduce the flexural moment versus rotation angle between the column and the beam member. Then, the influence of number and location of weld spots and the beam-to-column connection characteristics on the structural behaviour of a beam member is also analysed through nonlinear GBT simulations. In fact, GBT can be a useful numerical alternative to determine the structural response of pallet rack systems taking into account several of its particularities, such as weld spots and sectional deformation, with a low computational cost. Finally, GBT results are compared with shell finite element values.

2 Generalised Beam Theory formulation

2.1 Overview of GBT geometrical nonlinear analysis

Two different steps are required to perform a GBT calculation: a cross-section analysis, where the deformation modes and classical GBT tensors are found, and the member analysis. The deformation modes obtained after the cross-section analysis are categorised in four different families depending on its kinematics: Conventional modes (involve warping and in-plane displacements and assume the Vlasov hypotheses), Natural Shear modes (only involve in-plane displacements), Transverse Extension modes (only involve in-plane displacements) and Local Shear modes (only involve warping displacements). The displacement field, according to the local coordinate system of each sectional wall (Fig. 1), can be obtained through Eq. 1 and 2, where k is the deformation mode and u_k, v_k, w_k and ϕ_k correspond to the cross-section shape functions and modal amplitude functions; respectively.

$$\begin{bmatrix} U_x \\ U_s \\ U_z \end{bmatrix} = \begin{bmatrix} u(x,s) - z \cdot w,_x (x,s) \\ v(x,s) - z \cdot w,_s (x,s) \\ w(x,s) \end{bmatrix} \quad (1)$$

$$u(x,s) = \sum_{k=1}^{N} u_k(s)\phi_{k,x}(x) \quad v(x,s) = \sum_{k=1}^{N} v_k(s)\phi_k(x) \quad w(x,s) = \sum_{k=1}^{N} w_k(s)\phi_k(x) \quad (2)$$

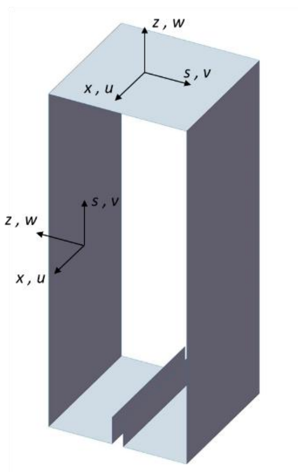

Figure 1 Local coordinate system defined for each sectional wall [21]

In this paper, the simplified formulation presented by Martins et al. [19] is used to perform the GBT nonlinear analysis. A brief overview of this formulation is presented here.

First of all, the whole set of Green-Lagrange membrane strains and small-strain bending effects are adopted. As a result, the strain-displacement relationships can be calculated through Eq. 3-5.

$$\varepsilon_{xx} = u_{,x} - z w_{,xx} + \frac{1}{2}\left(u_{,x}{}^2 + v_{,x}{}^2 + w_{,x}{}^2\right) \quad (3)$$

$$\varepsilon_{ss} = v_{,s} - z w_{,ss} + \frac{1}{2}\left(u_{,s}{}^2 + v_{,s}{}^2 + w_{,s}{}^2\right) \quad (4)$$

$$\varepsilon_{xs} = u_{,s} + v_{,x} - 2z w_{,xs} + u_{,x}u_{,s} + v_{,x}v_{,s} + w_{,x}w_{,s} \quad (5)$$

A linear and isotropic material behaviour is assumed; therefore, the constitutive material model described in Eq. 6 is used, where E and v are the material Young's modulus and Poisson's ratio; respectively.

$$\begin{bmatrix} \sigma_{xx} \\ \sigma_{ss} \\ \sigma_{xs} \end{bmatrix} = \frac{E}{1-\upsilon^2} \begin{bmatrix} 1 & \upsilon & 0 \\ \upsilon & 1 & 0 \\ 0 & 0 & \frac{1-\upsilon}{2} \end{bmatrix} \begin{bmatrix} \varepsilon_{xx} \\ \varepsilon_{ss} \\ \varepsilon_{xs} \end{bmatrix} \quad (6)$$

The member strain energy can be calculated combining Eq. 3-7. The addition of an initial geometrical imperfection has not been considered for the cases of study presented in Section 4.

$$U = \frac{1}{2}\int_V \sigma_{ij}\varepsilon_{ij}\, dV = U_1 + U_2 + U_3 \quad (7)$$

A 2-node beam element is used to solve the member equilibrium equations. The degrees of freedom of the GBT beam element correspond to the values of the amplitude functions or amplitude functions derivatives. These modal amplitude functions are described by Hermite cubic polynomials for the conventional modes, natural shear modes and transverse extension modes. Conversely, Lagrange cubic polynomials are used for the deformation modes that only involve warping displacements (axial extension and local shear modes).

The finite element internal force (Eq. 8) and the tangent stiffness matrices components (Eq. 9) are calculated by differentiating the strain energy and the internal force vector; respectively. More details about the GBT formulation can be found in [19].

$$f_h^{(e)} = \frac{\delta U^{(e)}}{\delta d_h} = f_h{}^1 + f_h{}^2 + f_h{}^3 \quad (8)$$

$$T_{hi}^{(e)} = \frac{\delta f_h^{(e)}}{\delta d_i} T_{hi}^1 + T_{hi}^2 + T_{hi}^3 \quad (9)$$

Finally, an incremental-iterative procedure based on the Newton-Raphson's algorithm has been used to solve the nonlinear analysis.

2.2 Introduction of sectional constraints

Sectional constraint equations can be used to reproduce weld spots on beam members [21]. The same global displacements and in-plane rotation (Eq. 10-12) are imposed at the sectional nodes where weld spots are located by means of constraint equations.

$$\sum_k \left(\left(u_k(s_i) - u_k(s_j)\right)\phi_{k,x}(L_s)\right) = 0 \quad (10)$$

$$\sum_k \left(\left([R]\begin{bmatrix} v_k(s_i) \\ w_k(s_i) \end{bmatrix} - [P]\begin{bmatrix} v_k(s_j) \\ w_k(s_j) \end{bmatrix} \right) \phi_k(L_s) \right) = \begin{bmatrix} 0 \\ 0 \end{bmatrix} \tag{11}$$

$$\sum_k \left(\left(w_{k,s}(s_i) - w_{k,s}(s_j) \right) \phi_k(L_s) \right) = 0 \tag{12}$$

Where s_i and s_j are the *s* coordinate positions of the two welded sectional nodes, L_s is the longitudinal coordinate of the cross-section with the weld spot and *[R]* and *[P]* are the transformation matrices from local to global coordinate system for the sectional wall *r* and *p*; respectively.

These additional constraint equations are added into the global system by means of Lagrange multiplier's method.

A minimum number of deformation modes must be included into the analysis when these sectional constraint equations are added into the model. In fact, the authors recommend the addition of symmetric and asymmetrical deformation modes; otherwise, the global stiffness matrix after the addition of sectional constraint equations could be close to be singular.

2.3 Nonlinear pure modal springs

It is well known that a typical beam-to-column connection in a pallet rack structure shows a nonlinear moment-rotation curve. In fact, finite element analyses commonly use nonlinear springs in order to reproduce these types of connections [1-2].

In this paper, a 1-dimensional nonlinear pure modal spring is used to describe the beam-to-column connection. A cubic polynomial equation has been introduced to define the nonlinear flexural moment – rotation relationship into the GBT formulation (Eq. 13). Thus, the spring strain energy can be calculated by means of Eq. 14. After combining Eq. 8, 9 and 14, the internal force components and the 2x2 elemental tangent stiffness matrices are given by Eq. 15-19.

$$M = a \cdot u^3 + bu^2 + cu \tag{13}$$

$$U = \frac{1}{4}a(u_2 - u_1)^4 + \frac{1}{3}b(u_2 - u_1)^3 + \frac{1}{2}c(u_2 - u_1)^2 \tag{14}$$

$$f_1^e = a\left(-u_2^3 + 3u_1u_2^2 - 3u_1^2u_2 + u_1^3\right) + b\left(-u_2^2 + 2u_1u_2 - u_1^2\right) + c(u_1 - u_2) \tag{15}$$

$$f_2^e = a\left(u_2^3 - 3u_1u_2^2 + 3u_1^2u_2 - u_1^3\right) + b\left(u_2^2 - 2u_1u_2 + u_1^2\right) + c(u_2 - u_1) \tag{16}$$

$$\left[T^1\right]^e = c\begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \tag{17}$$

$$\left[T^2\right]^e = 2b\begin{bmatrix} u_2 - u_1 & u_1 - u_2 \\ u_1 - u_2 & u_2 - u_1 \end{bmatrix} \tag{18}$$

$$\left[T^3\right]^e = 3a\begin{bmatrix} u_2^2 - 3u_1u_2 + 3u_1^2 & -u_2^2 + 3u_1u_2 - u_1^2 \\ -u_2^2 + 3u_1u_2 - u_1^2 & u_2^2 - 3u_1u_2 + 3u_1^2 \end{bmatrix} \tag{19}$$

Where *a*, *b* and *c* are the coefficients of the cubic polynomial equation and u_1 and u_2 are the degrees of freedom (rotation angle) of the two nodes of the 1-dimensional spring.

The components of element internal force vector and tangent stiffness matrices can be directly assembled into the global system as it is a pure modal spring.

3 Numerical analysis

3.1 Structural characteristics

The analysed beam member is a rectangular monosymmetric open cross-section with the following main dimensions: 50 mm width, 130 mm height and 2 mm of thickness (Fig. 2(a)). The beam member has weld spots along its length as shown in Fig. 2(b), in order to reduce the distortional deformation under a flexural load.

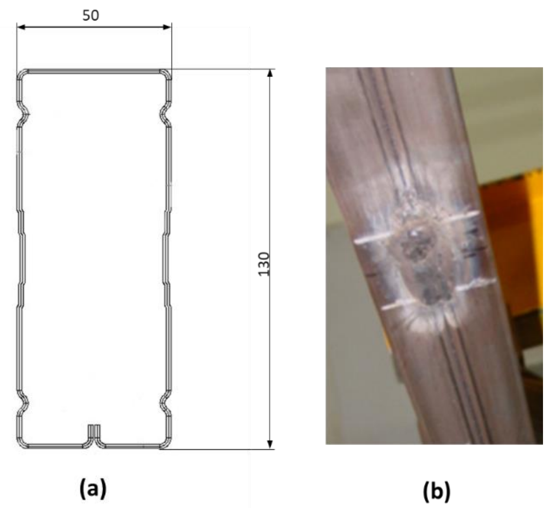

Figure 2 (a) Main dimensions of the analysed beam section. (b) Detail of a sectional weld spot between the sectional lips.

An isolated beam member of 2600 mm length with 4 concentrated loads (Fig. 3) has been used to compare shell and GBT finite element results and validate the introduction of weld spots and nonlinear modal springs into GBT second order analysis. Furthermore, the same load case is used to evaluate the influence of number of weld spots as well as the influence of the beam-to-column connection characteristic on the beam distortional deformation.

The nonlinear beam-to-column connection behaviour, which depends on the characteristic of the beam, the column and the connection system, is experimentally obtained through the test method described in the European Standard EN15512 [22] in Annex A.2.4. The experimental set-up and test results are shown in Fig. 4(a). The moment-rotation curve is obtained for the connection between the analysed beam cross-section and a column with a high load carrying capacity. Moreover, a multilinear and a cubic polynomial curves have been adjusted to the experimental relationship, as shown in Fig. 4(b) in order to define the elemental spring behaviour in the shell and GBT analysis; respectively. In addition, it has been assumed that the beam-to-column connection completely restrains the axial, torsional and sectional deformations at the end cross-sections.

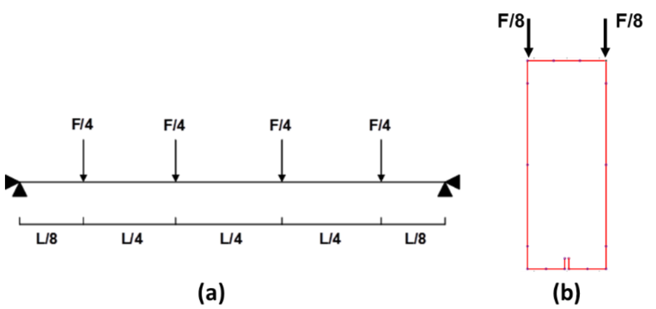

Figure 3 (a) Schematics of the analysed load case with 4 concentrated forces. (b) Detail of the application of concentrated forces at one cross-section.

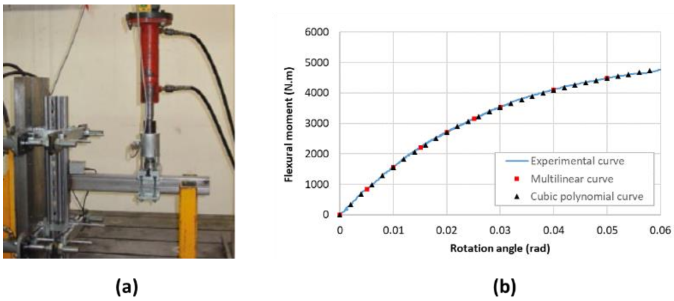

Figure 4 (a) Experimental test to determine the beam-to-column connection behaviour. (b) Comparison of the experimental, multilinear and cubic polynomial moment- rotation curves.

3.2 Shell finite element model

All shell finite element analyses presented in this paper are done by ANSYS software. A 4-node shell finite element (SHELL 181) with reduced integration is used to mesh the beam member. The weld spots are simulated by means of couplings of all degrees of freedom between the two nodes where the weld spots are located. Rigid end plates have been included at both beam ends to restrain the sectional and warping displacements. Furthermore, the displacements and two rotations have been restraint at the node located at the centre of gravity of the beam cross-section at each end plate (Fig. 5). Finally, a 1-dimensional nonlinear spring (COMBIN39) with the multilinear curve (Fig. 4(b)) is introduced to reproduce the beam-to-column connection characteristics.

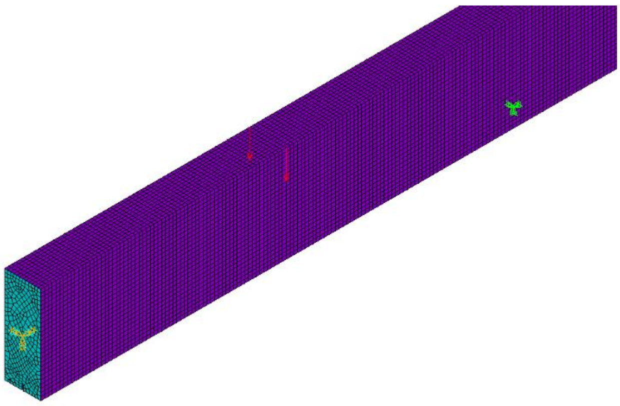

Figure 5 Detail of the shell finite element model. An element of 5 x 5 mm size has been used for the member discretization.

3.3 GBT finite element model

The beam cross-section has been discretized with 18 sectional nodes as shown in Fig. 6. Even though 54 deformation modes can be obtained after the cross-section analysis (the quadratic transverse modes are not considered), only 10 deformation modes (Fig. 7) are included in the GBT nonlinear analysis. 32 GBT beam elements have been used for the longitudinal discretization.

A pure modal nonlinear spring has been defined to reproduce the beam-to-column connection. In this case, the formulation presented in Section 2.3 is used to describe the moment-rotation relationship. On the other hand, the weld spots are introduced through the sectional constraint equations defined in Section 2.2 between sectional nodes 2 and 17 (Fig. 6).

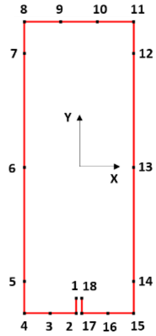

Figure 6 GBT Sectional discretization with 18 nodes. Global coordinate axes.

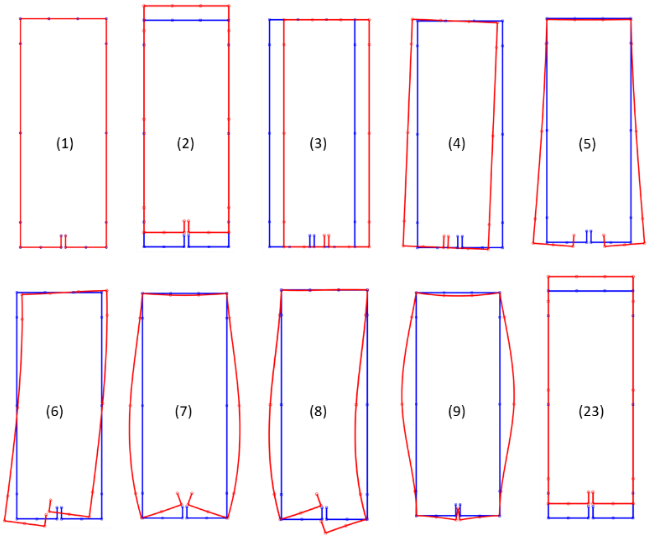

Figure 7 Deformation modes involved in the GBT nonlinear analysis (Conventional modes 1-9 and Natural shear mode 23).

4 Discussion of results

The influence of weld spots and the beam-to-column connection behaviour have been analysed by means of shell and GBT nonlinear finite element analyses. In both cases, the Newton-Raphson's method has been used to perform the geometrical nonlinear analysis.

4.1 Influence of weld spots

Four different cases have been simulated in order to evaluate the influence of weld spots considering the beam-to-column connection behaviour presented in Fig. 4(b). The schematics of each case are shown in Fig. 8. The differences between them are the number and location of weld spots (without and with 1, 2 or 3 weld spots). A total force (F) of 80000N has been applied in all cases.

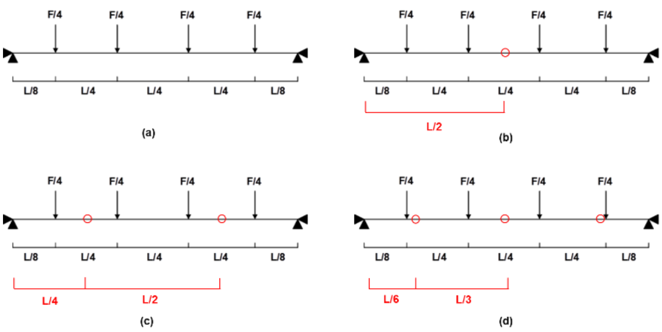

Figure 8 (a) Load case without weld spots and with (b) 1, (c) 2 and (d) 3 weld spots, respectively. The red circles correspond to longitudinal positions of weld spots.

First of all, it is a worth mentioning that a good agreement is obtained between shell finite element analysis and GBT. Furthermore, it is noted that it is not necessary to add a large number of deformation modes to accurately reproduce the weld spots by means of the constraint equations, which it is important in terms of computational cost. The displacement field of shell and GBT simulations for the beam without weld spots is presented in Fig. 9 and 10. On the other hand, the results for the beam with two weld spots are shown in Fig. 11 and 12. It can be observed that weld spots reduce the displacement field of the beam member, especially the *x* displacement (global coordinate axes), which it is related to the distortional deformation. Conversely, the bending deflection in both cases is similar.

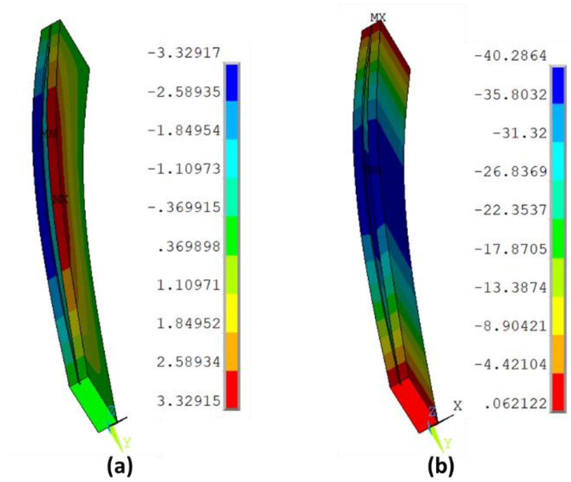

Figure 9 (a) Horizontal (*x*) and (b) vertical (*y*) displacement obtained for the shell analysis of the beam member without weld spots.

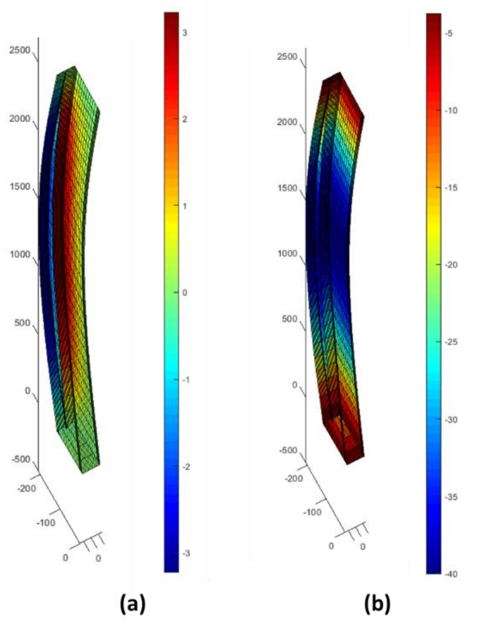

Figure 10 (a) Horizontal (*x*) and (b) vertical (*y*) displacement obtained for the GBT analysis of the beam member without weld spots.

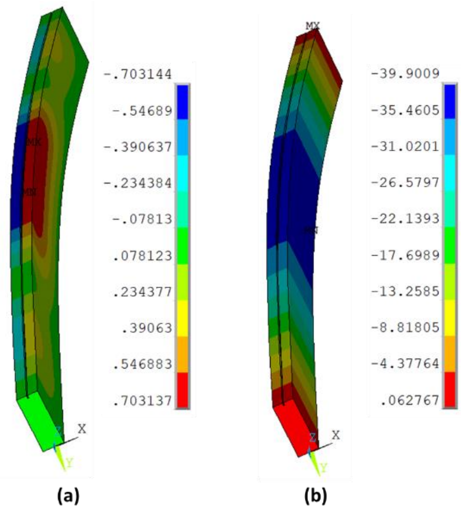

Figure 11 (a) Horizontal (*x*) and (b) vertical (*y*) displacement obtained for the shell analysis of the beam member with two weld spots.

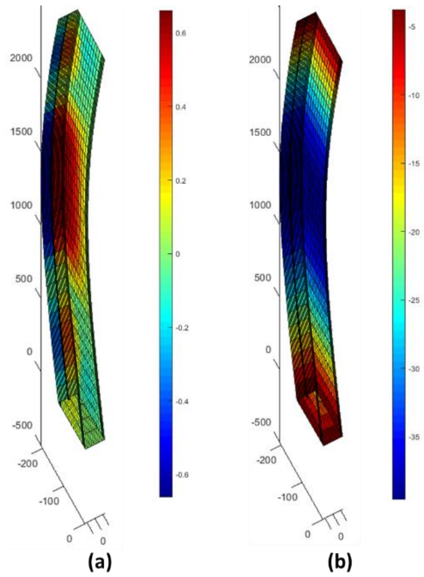

Figure 12 (a) Horizontal (*x*) and (b) vertical (*y*) displacement obtained for the GBT analysis of the beam member with two weld spots.

The *x* displacement at the sectional node 4 (Fig. 6) along the beam length is presented for the four different cases in Fig. 8. It can be observed how the *x* displacement decreases as the number of weld spots increases. In fact, with 2 weld spots the maximum horizontal displacement is about five times lower than the case without weld spots (Fig. 13). In addition, the maximum horizontal displacement is achieved at a different longitudinal coordinate according to the number of weld spots. Consequently, the real distribution of weld spots should be considered in order to proper reproduce the distortional deformation of this type of beam member.

Finally, the results show that the influence of weld spots on the vertical deflection is practically negligible.

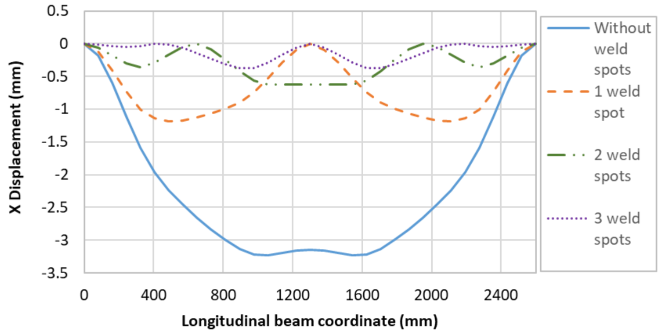

Figure 13 Comparison of the x displacement of sectional node 4 along the member length for the four different cases obtained through GBT analyses.

4.2 Influence of beam-to-column connection behaviour

The beam-to-column connection behaviour depends on the geometrical characteristics of the beam and the column. The joint behaviour presented at Section 3.1 corresponds to a column with a high load carrying capacity. In this section, three different types of connections are analysed in order to evaluate its influence on the structural response of the beam member. As a first case, a perfectly rigid column and beam-to-column connection is assumed; therefore, both beam ends are clamped. Secondly, the flexural moment versus rotation angle presented in Section 3.1 is considered (*S1*). Finally, the experimental nonlinear flexural moment versus rotation curve obtained for the same beam connected to a column with a lower load carrying capacity (*S2*) is used. This connection presents a lower stiffness as shown in Fig. 14. In each semi rigid connection, a cubic polynomial equation (Section 2.3) has been adjusted to the experimental curve in order to introduce a pure modal spring in the GBT nonlinear analysis. The comparison between curves is also shown in Fig. 14.

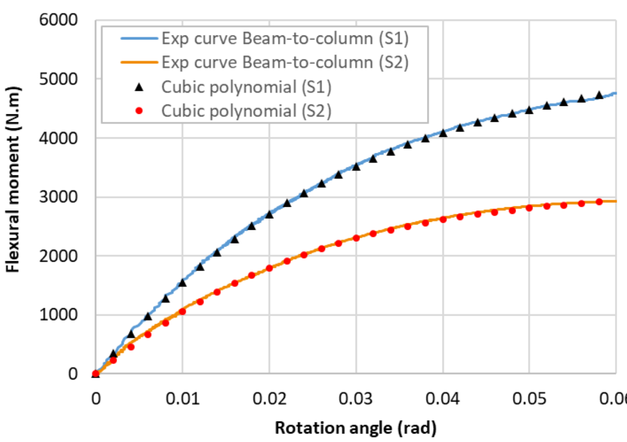

Figure 14 Comparison of flexural moment versus rotation angle for the beam connected to a column with a high (*S1*) and a medium (*S2*) load carrying capacity.

All three cases of simulation have been done considering that the beam member has two weld spots along its length in order to evaluate the influence of the beam-to-column connection characteristics. First of all, the *x* displacement values of sectional node 4 located at the middle section of the beam member for the three different end boundary conditions are analysed. Fig. 15 shows the displacement results according to the total applied external force. It can be observed that higher *x* displacements are obtained for stiffer connections.

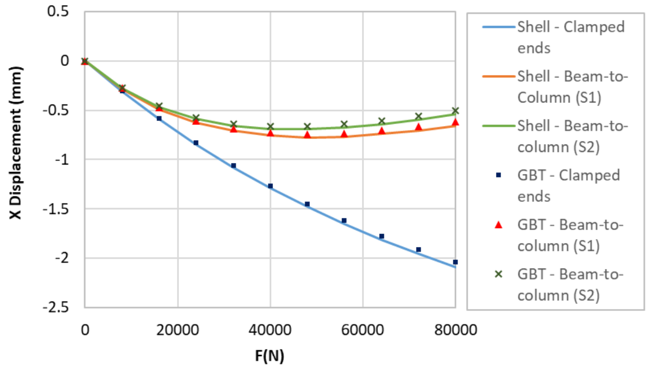

Figure 15 X displacement of sectional node 4 at the middle beam section obtained through shell and GBT analysis for clamped ends, a connection to a column with high load carrying capacity (S1) and a connection to a column with a medium load carrying capacity (S2)

Furthermore, the characteristics of the beam end conditions have also an effect on the reaction forces. The axial and flexural moment reactions are compared in Fig.16 and 17; respectively. Although the *x* displacement is higher for the load case with clamped ends, the axial reaction is lower than the other cases because of the lower vertical deflection. On the other hand, Fig. 18 shows the moment reaction – applied force curves. The differences between each case are clearly visible. Moreover, the nonlinear behaviour of the moment reaction is obtained when the pure spring is introduced into the GBT model in order to reproduce the beam-to-column connection. Consequently, it is important to define the real nonlinear relationship of the beam-to-column connection to obtain accurate reactions, internal force diagrams and displacement values.

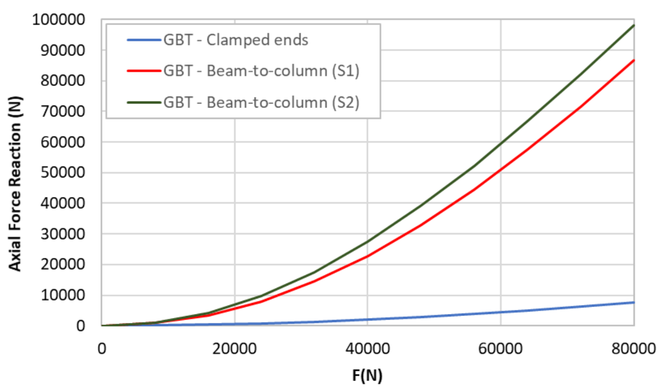

Figure 16 Axial force reaction obtained through GBT analysis for clamped ends, a connection with a column with high load carrying capacity (*S1*) and a connection with a column with a medium load carrying capacity (*S2*).

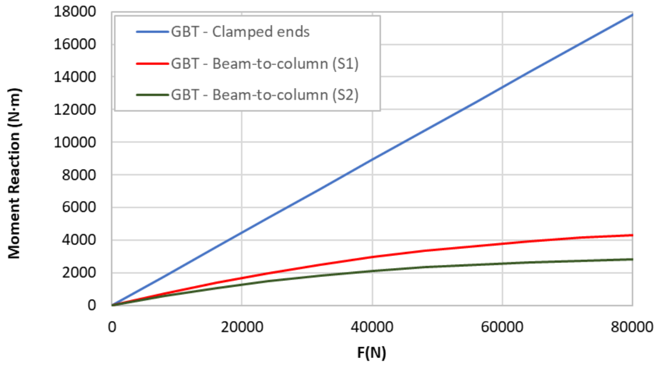

Figure 17 Moment reaction obtained through GBT analysis for clamped ends, a connection with a column with high load carrying capacity (*S1*) and a connection with a column with a medium load carrying capacity (*S2*).

5 Conclusions

This paper presents a methodology to introduce weld spots and the beam-to-column connection behaviour in a GBT geometrical nonlinear analysis of a pallet rack beam. A proper introduction of these two characteristics is necessary in order to obtain accurate results in a finite element analysis. Therefore, the use of advanced beam models, which can handle these types of issues, as well as sectional deformations, arises as a useful alternative to traditional shell finite element analysis. In fact, GBT results have been successfully compared with shell finite element values.

The introduction of weld spots is done by means of sectional constraint equations, which impose the same global displacements and the same in-plane rotation to the coupled sectional nodes. Moreover, it is possible to obtain accurate values with a few deformation modes. However, the introduction of a minimum number of symmetric and asymmetric deformation modes into the GBT analysis is recommended to obtain a non-singular reduced stiffness matrix when the additional constraint equations are added into the model.

On the other hand, the use of a cubic polynomial is enough to accurately reproduce the beam-to-column connection behaviour. Consequently, a nonlinear pure modal spring can be easily introduced into the GBT analysis to take into account the main connection characteristics.

The results have demonstrated the influence of weld spots in the displacement field of the beam member under a flexural load. The introduction of a small number of weld spots drastically reduces the distortional deformation of the beam member. Consequently, the use of weld spots is recommended when open cross-sections are employed as beam members in pallet rack structures in order to minimize its distortional deformation.

Additionally, the beam-to-column connection behaviour also have influence on the distortional deformation of the beam member. Higher distortional deformations are found for stiffer beam-to-column connections.

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