

Stress–Strain Behaviour Of A Pultruded Gfrp Frame for A Low-Rise 3d-Printed Building Accounting for Joint Flexibility and Shear Deformation

A Parametric Finite Element Study

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Abstract

The stress–strain behaviour of a two-storey load-bearing frame for a low-rise building, comprising pultruded glass-fibre-reinforced polymer (GFRP) hollow sections and intended for use with non-load-bearing 3D-printed walls, is investigated. The study aims to quantify the effects of joint rotational flexibility, transverse shear and the bracing system on frame displacements, internal forces, elastic stability and dynamic characteristics. A two-dimensional finite element model of a three-bay, two-storey frame measuring 10.8×6.0 m is developed. The columns, beams and diagonal braces have cross-sections of $200 \times 200 \times 12$ mm, $300 \times 150 \times 12$ mm and $100 \times 100 \times 8$ mm, respectively. Members are represented by Timoshenko beam theory, beam-to-column connections by rotational springs, and geometric stability by a generalised eigenvalue problem. Braced and unbraced configurations are analysed over a range of joint stiffnesses, from pinned to idealised rigid connections. At the baseline stiffness $k\theta = 15$ MN·m/rad, the roof displacement under the adopted storey-force pattern is 59.44 mm for the unbraced frame and 2.87 mm for the braced frame; the maximum interstorey drift ratio decreases from 10.76 to 0.51 ‰. The critical load factor increases from 10.49 to 80.41, while the fundamental vibration period decreases from 1.040 to 0.228 s. Under a uniformly distributed load of 12.6 kN/m, the calculated deflection of an isolated beam with $k\theta = 15$ MN·m/rad is 3.82 mm, of which 1.03 mm, or 26.9%, is attributable to transverse shear. The bracing subsystem is shown to virtually eliminate the sensitivity of global displacement to uncertainty in the rotational stiffness of frame joints, whereas assuming rigid connections in the unbraced configuration can substantially distort the results. Requirements for the analytical model and multilevel experimental verification are formulated.

Keywords: Pultruded GFRP; glass-fibre-reinforced polymer; composite frame; stress–strain behaviour; finite element method; semi-rigid joint; rotational stiffness; shear deformation; stability; interstorey drift ratio.

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1. Introduction

Pultruded GFRP profiles combine low weight, corrosion resistance and high specific strength; however, their structural efficiency is not governed solely by material strength limits. The longitudinal elastic modulus of GFRP is generally considerably lower than that of steel, its transverse and shear moduli are lower still, and thin section walls and local connection regions are susceptible to buckling, bearing failure and splitting. Consequently, displacements, local stability or connection behaviour often govern the limit state of a frame, rather than attainment of the longitudinal fibre strength [1, 3–9].

The publication of ASCE/SEI 74-23 and CEN/TS 19101:2022 has established a contemporary code framework for the design of pultruded composite structures [1–4]. Nevertheless, checking an individual member against a code does not remove the need to model the frame correctly as a system. Experimental studies of frame connections show that joints traditionally described as pinned or rigid actually exhibit finite rotational stiffness, an initial slip region and a nonlinear moment–rotation relationship [20–29]. An incorrect joint idealisation changes moment distribution,

lateral stiffness, column effective lengths and natural periods.

This issue is particularly important for a hybrid building with non-load-bearing 3D-printed walls. The printed envelope must not inadvertently act as a strut, because cracking, shrinkage and construction joints make such behaviour uncertain. Global stability must be provided by the composite frame itself, through bracing, floor diaphragms and connections with quantified structural properties. The analytical model must therefore distinguish the load-bearing and envelope subsystems, accounting for wall interaction only through explicitly designed out-of-plane restraints and movement clearances.

This paper addresses the second level of investigation of the proposed structural system: quantitative analysis of the frame. Unlike a preliminary check of an isolated column and beam, the present analysis simultaneously considers the interaction of two storeys and three bays, joint flexibility, transverse shear, the presence or absence of diagonal bracing, geometric stability and dynamic stiffness. All results relate to a demonstrative model and are intended to identify trends, not to establish ready-to-use design resistances.

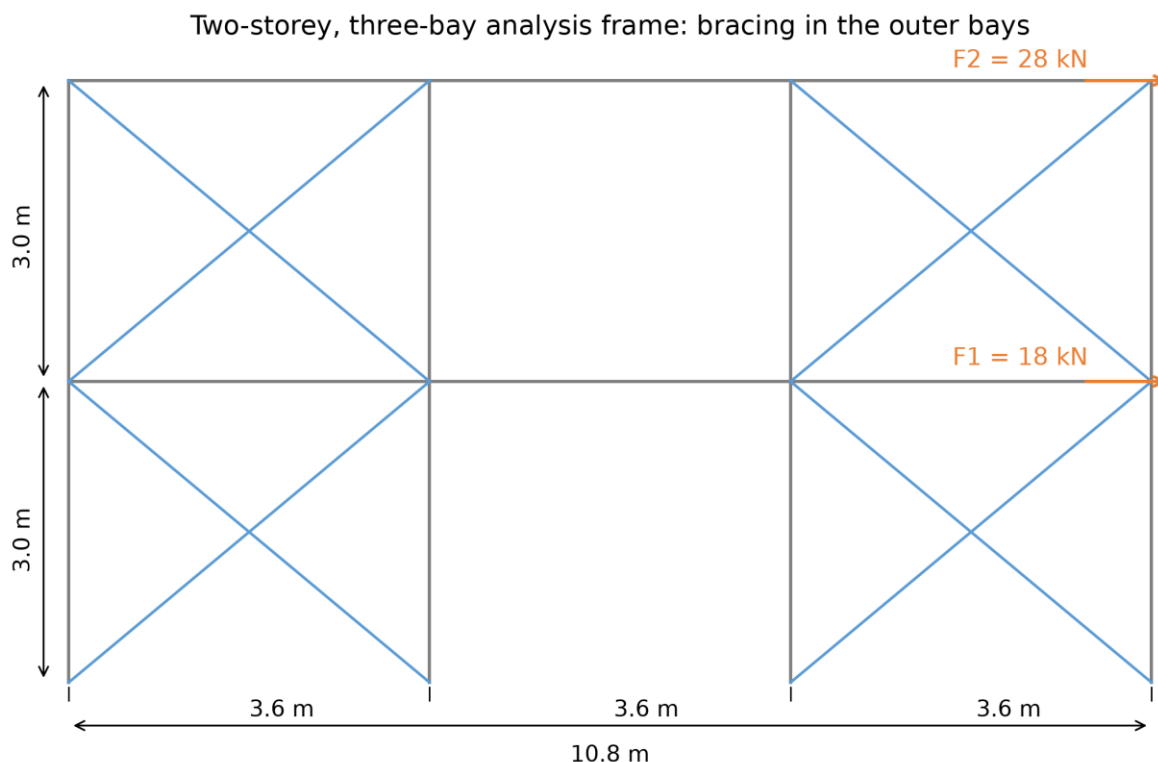


Figure 1 — Analytical configuration of the two-storey, three-bay frame and adopted horizontal storey forces

2. State of the art and research gap

2.1. Stability of pultruded columns

Experimental programmes on pultruded columns identify transitions between material failure, local buckling of section walls, and global flexural or flexural-torsional buckling. The observed mode depends on member length, plate width-to-thickness ratio, boundary conditions, initial curvature, reinforcement non-uniformity and shear modulus [7–15]. A study of channel sections of different lengths showed that short specimens may fail by crushing or local buckling, whereas more slender members undergo global buckling [9]. This precludes determining load-bearing capacity solely from Euler's formula or coupon strength.

Recent calibrations of design models emphasise the importance of statistical variability in geometry and properties. For the local stability of I- and H-sections, CEN/TS 19101 employs separate flange- and web-buckling models, while reliability research demonstrates the need to align partial factors with resistance uncertainty [13]. For a building design model, this means that global analysis must provide member forces and effective lengths, but the final section verification must follow a qualified code procedure using data from the specific manufacturer.

2.2. Bending, transverse shear and serviceability

The low shear modulus of GFRP produces a significant shear contribution to deflection, particularly for deep sections and low span-to-depth ratios. Beam tests and analytical studies show that the Euler-Bernoulli model can underestimate deformation; shear also influences critical loads in the stability analysis of open sections [16–19]. In a frame, errors at individual-span level translate into joint rotations, moment redistribution and vertical floor displacements.

In addition to instantaneous deformation, creep, temperature and moisture are significant for the polymer matrix. The present study examines the short-term elastic response, but the methodology deliberately separates instantaneous results from load-duration factors. This distinction enables reduced moduli or rheological functions to be introduced subsequently without changing the model topology.

2.3. Semi-rigid connections

Tests on bolted, bonded-and-bolted and steel-plate-reinforced joints demonstrate a wide range of initial stiffnesses and ultimate moments [20–25]. A connection classified as nominally pinned transfers a finite moment; flange reinforcement and changes in bolt diameter, cleat thickness, hole clearance and bolt pretension alter its

moment–rotation curve [26–29]. Three-dimensional modelling with damage criteria provides further insight into local delamination. For global building analysis, however, it is generally appropriate to replace the joint with an equivalent rotational spring whose parameters are obtained from tests or a verified detailed model.

The system-level research gap is the lack of a quantitative comparison of three factors within a single

low-rise frame model: connection flexibility, member transverse shear and an explicitly designed bracing subsystem. Frequently, one factor is investigated while the others are idealised. For a hybrid 3D-printed building, it is necessary to establish which parameters actually govern the global response and where increased model complexity affects the engineering decision.

Table 1 — Research gap and adopted approach

Issue	Consequence of simplification	Approach adopted in this study
Joints assumed exclusively pinned or rigid	Distorted moments, drift and column effective lengths	$k\theta$ varied from 0 to an idealised infinite value
Transverse shear neglected	Deflection underestimated and stiffness overestimated	Timoshenko beam elements and separate deflection components
Printed walls used as implicit struts	Uncertain global stiffness after cracking	Walls excluded from the load-bearing model; frame provides stability
Bracing and moment-frame action not compared	Principal source of stiffness cannot be identified	Braced and unbraced models analysed
Material strength checked in isolation	Serviceability and stability limits overlooked	Stresses, drift ratio, period and α_{cr} considered
Numerical results reported without input-data status	Demonstrative estimates misinterpreted as design values	All parameters identified as replaceable and requiring testing

Note. The model does not replace local buckling and connection checks in accordance with ASCE/SEI 74-23 or CEN/TS 19101.

3. Aim, objectives and scientific novelty

The aim of the study is to establish the governing trends in the stress–strain behaviour of a pultruded GFRP frame for a low-rise 3D-printed building as joint rotational stiffness and bracing configuration are varied, accounting for transverse shear and geometric stability.

The following objectives were addressed:

- A parametric finite element model of a two-storey, three-bay frame incorporating beams, columns, diagonal braces and rotational springs was developed.
- The numerical implementation was verified against analytical solutions for a beam and a concentrically compressed column.
- Vertical deflections and moment distributions were determined as functions of $k\theta$.
- Lateral displacements and interstorey drift ratios were compared for braced and unbraced frames.
- Linear elastic buckling and free-vibration problems were solved.

- Calculated elastic stresses under the baseline combination and the sensitivity of the results to EL, GLT and brace area were assessed.
- Recommendations for analytical idealisation and experimental verification were formulated.

The scientific novelty lies in the combined parametric treatment of joint rotational flexibility and member shear deformability in a frame whose global stability is provided independently of the 3D-printed walls. A quantitative difference in sensitivity between braced and unbraced configurations is demonstrated: uncertainty in $k\theta$ has little effect on drift in the former, but can change roof displacement by almost an order of magnitude in the latter. An increasing share of shear deformation in total beam deflection as end-restraint stiffness increases is also established.

4. Methods

4.1. Geometry and analytical configurations

A two-dimensional longitudinal frame of a two-storey house is considered. Three 3.6 m bays give a total length

of 10.8 m; each storey is 3.0 m high. In the baseline configuration, X-bracing is installed in the two outer bays of each storey. The column bases are assumed fixed

to ensure comparability between configurations; the effect of base flexibility must be investigated separately once the base shoes and anchors have been developed.

Table 2 — Geometric parameters of the analytical model

Parameter	Value	Purpose
Number of storeys	2	Low-rise configuration
Number of bays	3	Assessment of internal-force redistribution
Span	3.6 m	Consistent with the building modular grid
Storey height	3.0 m	Residential use
Frame length	10.8 m	Three equal bays
Braced bays	Outer bays, both storeys	Independent lateral stiffness
Floor model	Rigid distribution of storey force among four nodes	Two-dimensional formulation
Contribution of printed walls	Excluded from global stiffness	Prevents unintended strut action

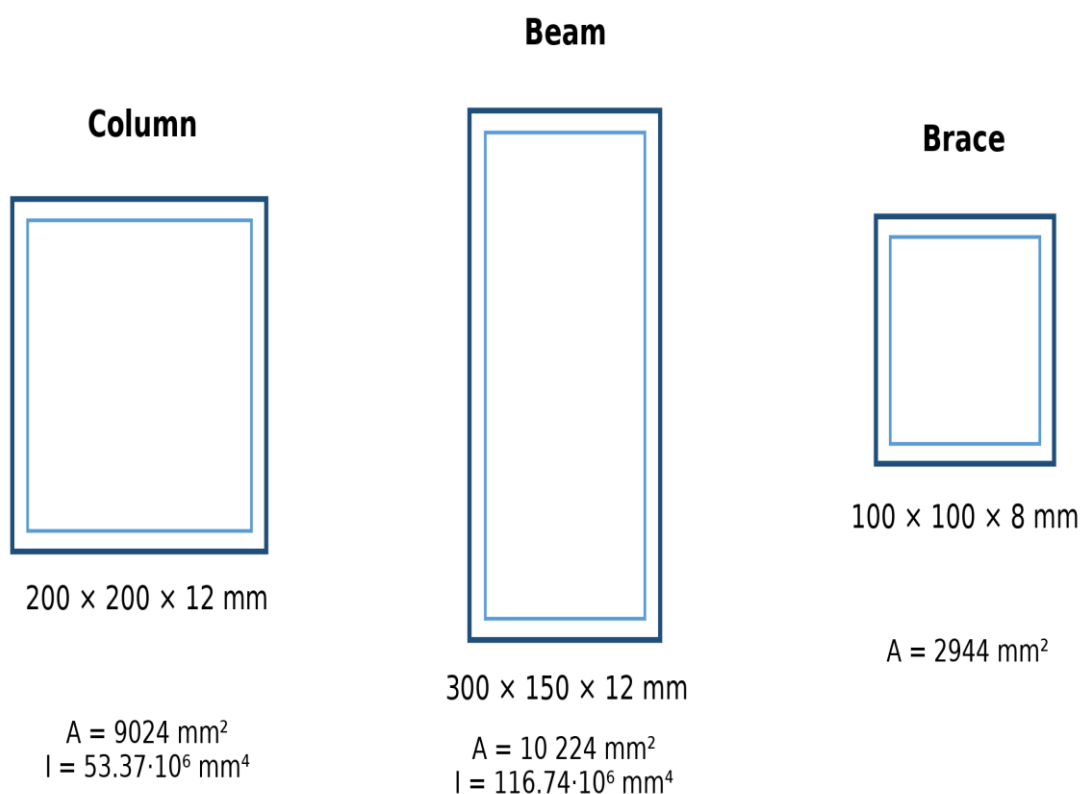


Figure 2 — Demonstrative hollow sections of the column, beam and diagonal brace

4.2. Cross-sections and material properties

The columns are assigned $200 \times 200 \times 12$ mm square hollow sections; the beams have rectangular hollow sections 300 mm deep, 150 mm wide and 12 mm thick; and the braces have $100 \times 100 \times 8$ mm square hollow

sections. Hollow sections are selected to reduce torsional sensitivity and simplify end protection. Calculated section properties are derived from nominal dimensions without allowance for corner radii or manufacturing non-uniformity.

Table 3 — Geometric properties of the profiles

Member	Section, mm	A, mm ²	In-plane I, mm ⁴	W, mm ³
Column	$200 \times 200 \times 12$	9 024	$53.37 \cdot 10^6$	533 740
Beam	$300 \times 150 \times 12$	10 224	$116.74 \cdot 10^6$	778 280
Brace	$100 \times 100 \times 8$	2 944	—	—

Note. Design verification must use the actual dimensions, corner radii, section-wall reinforcement and manufacturing tolerances.

The member material is assumed linear elastic. The longitudinal modulus EL is taken as 23 GPa and the shear modulus GLT as 3.0 GPa. These values fall within the range typical of structural pultruded GFRP profiles, but are used here for demonstration only. The transverse modulus ET = 8 GPa and Poisson's ratio of 0.25 are used

solely for a screening assessment of local buckling. A density of 1900 kg/m³ is adopted for mass estimation. The actual model must be based on longitudinal and transverse coupon tests, shear tests and batch certificates [38–40].

Table 4 — Adopted properties and data status

Parameter	Value	Status
Longitudinal modulus EL	23 GPa	Baseline parameter; varied over 18–28 GPa
Shear modulus GLT	3.0 GPa	Baseline parameter; varied over 2–4 GPa
Transverse modulus ET	8 GPa	Plate screening assessment only
Poisson's ratio ν_{LT}	0.25	Demonstrative value
Density	1900 kg/m ³	Self-mass estimation
Design resistances	Not assigned	Require code-based evaluation of test results
Temperature and load-duration factors	Not introduced	Subject of the next stage

4.3. Loads and combinations

Uniformly distributed line loads are applied to the beams at each level. The characteristic permanent component is $q_G = 8.1$ kN/m and the variable component is $q_Q = 4.5$ kN/m, giving a total of 12.6 kN/m for serviceability assessment. The demonstrative ultimate combination is

$q_{ULS} = 1.35q_G + 1.50q_Q = 17.685$ kN/m. The lateral system is subjected to storey forces $F_1 = 18$ kN and $F_2 = 28$ kN, distributed equally among the nodes at the respective levels. These forces do not constitute a code-based seismic assessment of a specific site; they provide a common action for comparing the stiffness of the configurations.

$$q_{ULS} = 1.35 q_G + 1.50 q_Q$$

(1)

Table 5 — Analysis cases

Designation	Actions	Response quantities
V-SLS	$q = 12.6 \text{ kN/m}$	Beam deflection, moments and shear contribution
V-ULS	$q = 17.685 \text{ kN/m}$	Axial forces and stability
H	$F1 = 18 \text{ kN}$; $F2 = 28 \text{ kN}$	Displacements, drift ratios and periods
V-ULS + H	Ultimate vertical loading and storey forces	Elastic stresses and internal forces
PARAM	Variation of k_θ , EL, GLT and brace area	Response sensitivity

Note. When adapting the model to a project, combinations must follow the applicable national codes and seismic actions must be based on site-specific parameters [35–37].

4.4. Finite element formulation

Columns and beams are represented by two-node planar Timoshenko elements with three degrees of freedom per node: axial displacement, transverse displacement and rotation. This formulation includes bending and

transverse shear. Diagonal braces are represented by truss elements that resist both tension and compression in a linear formulation. In a design model, slender unilateral braces require nonlinear deactivation of the compression diagonal or verification of its buckling resistance.

$$\phi_s = 12 \text{ EL I} / (\kappa \text{ GLT } A_v L^2) \quad (2)$$

where ϕ_s is the dimensionless shear-flexibility parameter, κ is the shear correction factor, and A_v is the effective shear area. In the baseline analysis, $\kappa = 1.0$; the beam shear area A_v is taken as the area of the two vertical webs, $2t(h - 2t)$, and that of a square column as $2t(b - 2t)$.

$$M = k_\theta (\theta_b - \theta_c) \quad (3)$$

The beam-to-column connection is modelled as a rotational spring between the beam-end rotation θ_b and the column-joint rotation θ_c . Compatibility is enforced in both translational directions. The rotational stiffness is varied over 0, 1, 5, 15 and 50 MN·m/rad, together with an idealised infinite value.

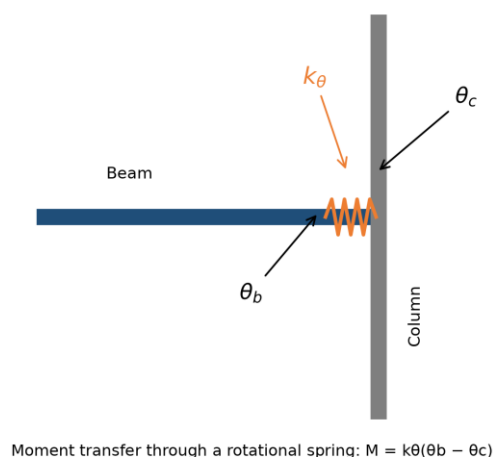


Figure 3 — Equivalent rotational spring for a semi-rigid beam-to-column connection

Internal beam-end rotations are introduced to incorporate the spring into the global matrix. Static condensation of

$$K^* = K_{ee} - K_{ei} K_{ii}^{-1} K_{ie} \quad (4)$$

where subscripts e and i denote the external nodal and internal end-rotation degrees of freedom, respectively. The uniformly distributed load is condensed using the same transformation, preventing inconsistency between stiffness and equivalent nodal forces.

$$\theta_i = (u_i - u_{i-1}) / h_i \quad (5)$$

For an isolated beam with identical rotational springs at both ends, the analytical support moment under a uniformly distributed load is expressed as

$$M_{end} = [k\theta q L^3 / (24 EI)] / [1 + k\theta L / (2 EI)] \quad (6)$$

The bending and shear components of midspan deflection are given by

$$\delta_b = 5 q L^4 / (384 EI) - M_{end} L^2 / (8 EI) \quad (7)$$

$$\delta_s = q L^2 / (8 \kappa GLT A_v) \quad (8)$$

Global elastic stability is assessed through a generalised eigenvalue problem using a geometric stiffness matrix assembled from the compressive forces in the columns:

$$\det (K - \alpha_{cr} KG) = 0 \quad (9)$$

An approximate second-order amplification factor for sensitivity assessment is written as $\eta = 1/(1 - 1/\alpha_{cr})$. It does not replace geometrically nonlinear analysis but

$$N_{cr,E} = \pi^2 EI / (KL)^2 \quad (10)$$

Modal analysis is performed for an equivalent two-mass system obtained by static condensation of the internal degrees of freedom with rigid floor diaphragms:

$$(K_f - \omega^2 M) \Phi = 0; \quad T = 2\pi/\omega \quad (11)$$

the internal degrees of freedom gives the effective element matrix:

4.5. Assessment criteria

The interstorey drift ratio is defined as the difference between the mean lateral displacements of adjacent levels divided by the storey height:

allows the influence of P-Δ effects to be classified. For an isolated concentrically compressed column, the Euler reference value is used:

Elastic normal stresses in the members are determined by superposition of axial force and bending:

$$\sigma_{\max} = |N|/A + |M|/W \quad (12)$$

Design resistances are deliberately not substituted into Eq. (12): their assignment depends on the characteristic properties of the specific profile, load-duration, temperature, environmental and member-size factors, and the adopted reliability format [1–4, 13, 33–35].

5. Verification of the numerical implementation

Before the parametric study, the numerical implementation was checked against two benchmark problems. For a simply supported beam under $q = 12.6$ kN/m, the analytical Euler–Bernoulli bending deflection is 10.26 mm, the shear contribution is 1.03 mm and the total is 11.29 mm. A discretisation of forty Timoshenko

elements reproduced the total value with a discrepancy below 0.2%. For idealised fixed ends, the support moment is $qL^2/12 = 13.61$ kN·m and the deflection including shear is 3.08 mm.

For a $200 \times 200 \times 12$ mm column of length 3.0 m with $K = 1.0$, the analytical critical load from Eq. (10) is 1346 kN. The single-element eigenvalue problem produced a value matching the analytical result to machine precision. These checks confirm the correctness of the matrix transformations; they do not validate local buckling, damage or connection models.

Table 6 — Verification of the numerical implementation

Benchmark problem	Analytical value	Numerical value	Discrepancy
Simply supported beam: total deflection	11.290 mm	11.307 mm	0.15%
Fixed-ended beam: support moment	13.608 kN·m	13.612 kN·m	0.03%
Column: $N_{cr,E}$	1346.2 kN	1346.2 kN	< 0.01%

Note. The small deflection discrepancy is associated with the interpolation scheme and equivalent nodal load of the Timoshenko element.

6. Results

6.1. Beam response to vertical loading and the effect of joint flexibility

At $k\theta = 0$, the beam is simply supported: the support moment is zero, the maximum span moment is 20.41 kN·m and the total deflection is 11.29 mm. Increasing $k\theta$ redistributes moment towards the supports and reduces deflection. At $k\theta = 15$ MN·m/rad, the support moment reaches 12.38 kN·m, the span moment is 8.03 kN·m and the deflection is 3.82 mm. Increasing the stiffness from 15 to 50 MN·m/rad provides only a further 0.50 mm

reduction in deflection; the response therefore rapidly approaches the fixed-end asymptote.

In the adopted model, the shear component is independent of the restraint moment because the shear-force diagram remains unchanged under a symmetric uniformly distributed load. Its relative contribution therefore increases from 9.1% for pinned ends to 26.9% at $k\theta = 15$ MN·m/rad and 33.4% for fixed ends. Paradoxically, the more effectively the joint suppresses bending deflection, the more pronounced the error associated with the Euler–Bernoulli model becomes.

Table 7 — Response of an isolated beam under $q = 12.6 \text{ kN/m}$

$k\theta$, $\text{MN}\cdot\text{m/rad}$	M_{support} , $\text{kN}\cdot\text{m}$	M_{span} , $\text{kN}\cdot\text{m}$	δ_b , mm	δ_s , mm	$\delta\Sigma$, mm	$\delta_s/\delta\Sigma$, %
0	0.00	20.41	10.26	1.03	11.29	9.1
1	5.46	14.95	6.97	1.03	7.99	12.8
5	10.48	9.93	3.94	1.03	4.97	20.7
15	12.38	8.03	2.80	1.03	3.82	26.9
50	13.21	7.20	2.29	1.03	3.32	31.0
Rigid	13.61	6.80	2.05	1.03	3.08	33.4

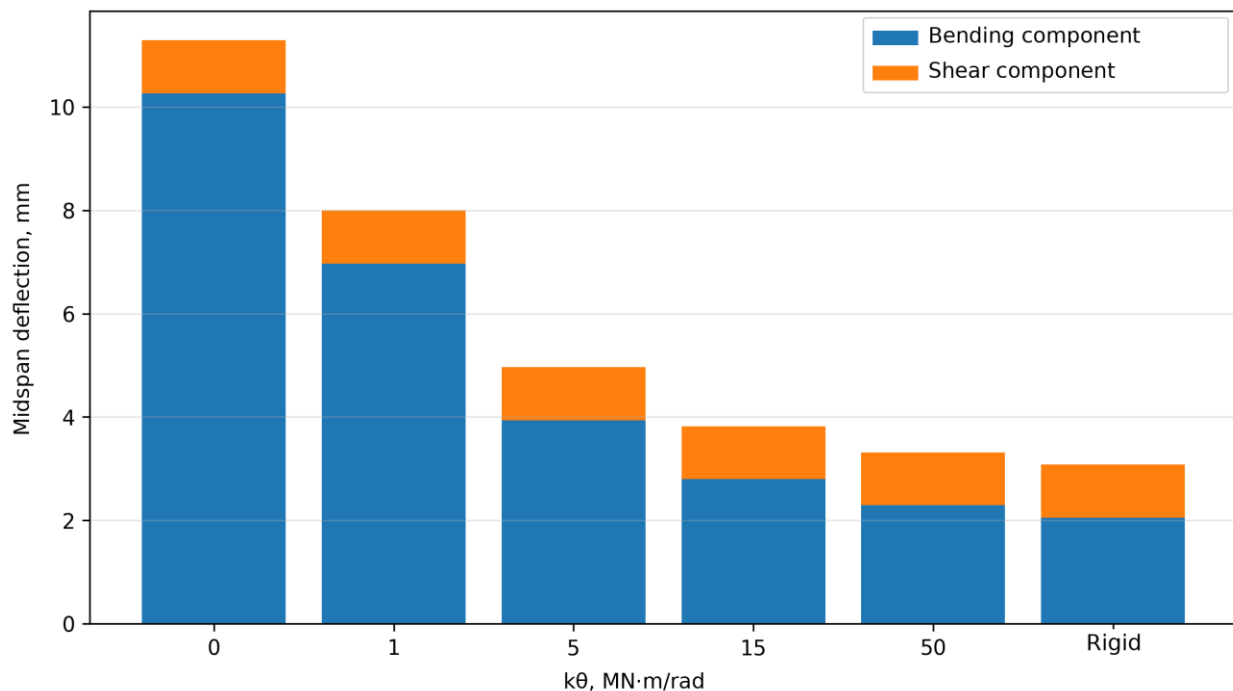


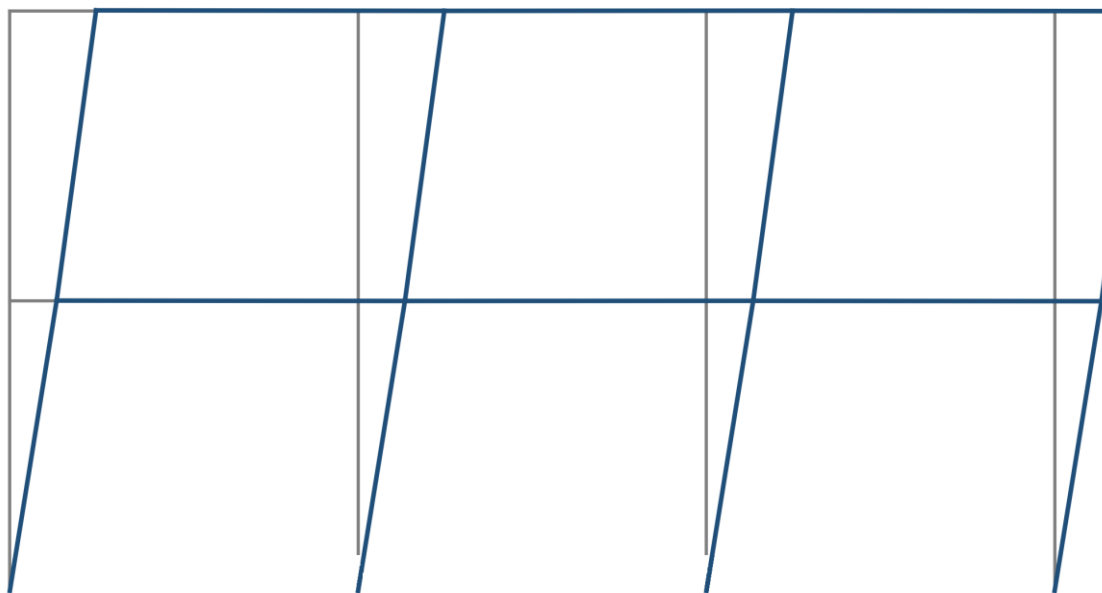
Figure 4 — Bending and shear components of beam deflection as joint rotational stiffness varies

This result has a direct design implication: classifying a joint solely by its moment-transfer capacity is insufficient. Serviceability assessment requires the actual initial portion of the moment-rotation curve and explicit consideration of GLT. When $k\theta$ is inferred from the ultimate moment or from secant stiffness at a large rotation, the calculated deflection may be underestimated.

6.2. Lateral displacements and the role of bracing

The most pronounced difference occurs between braced and unbraced configurations. At the baseline $k\theta = 15 \text{ MN}\cdot\text{m/rad}$, the frame without diagonals has a first-level displacement of 32.29 mm and a roof displacement of 59.44 mm. The storey drift ratios are 10.76 and 9.05 %. With X-bracing, the corresponding displacements decrease to 1.53 and 2.87 mm, and the drift ratios to 0.510 and 0.446 %. Roof displacement is reduced by a factor of 20.7.

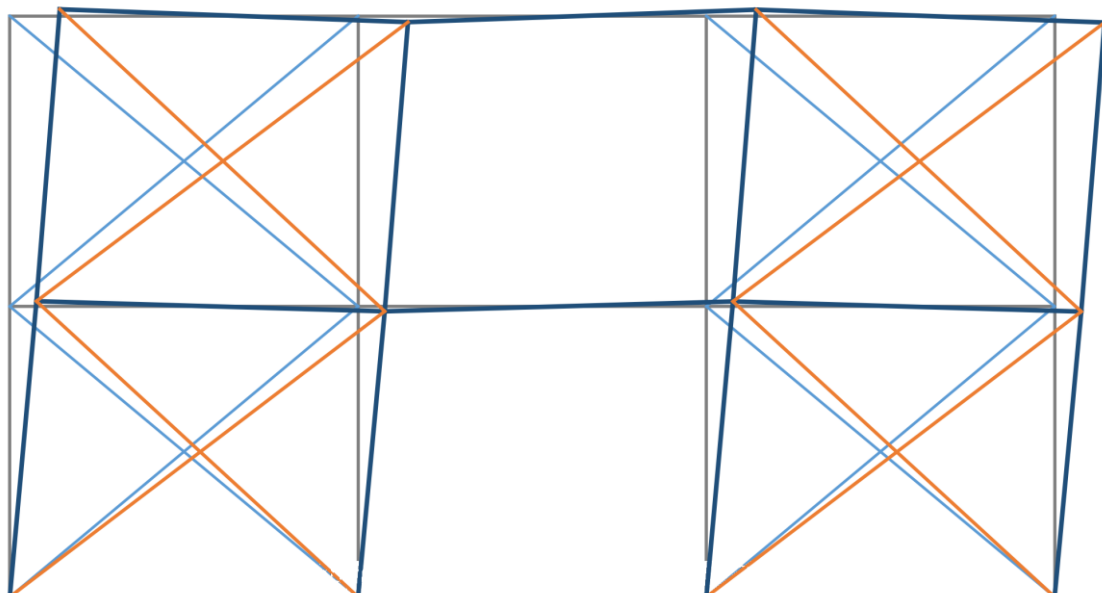
Unbraced frame



Displacements $\times 15$; roof displacement = 59.44 mm

Figure 5 — Deformed configuration of the unbraced frame at $k_0 = 15 \text{ MN}\cdot\text{m/rad}$

Braced frame



Displacements $\times 180$; roof displacement = 2.87 mm

Figure 6 — Deformed configuration of the braced frame at $k_0 = 15 \text{ MN}\cdot\text{m/rad}$

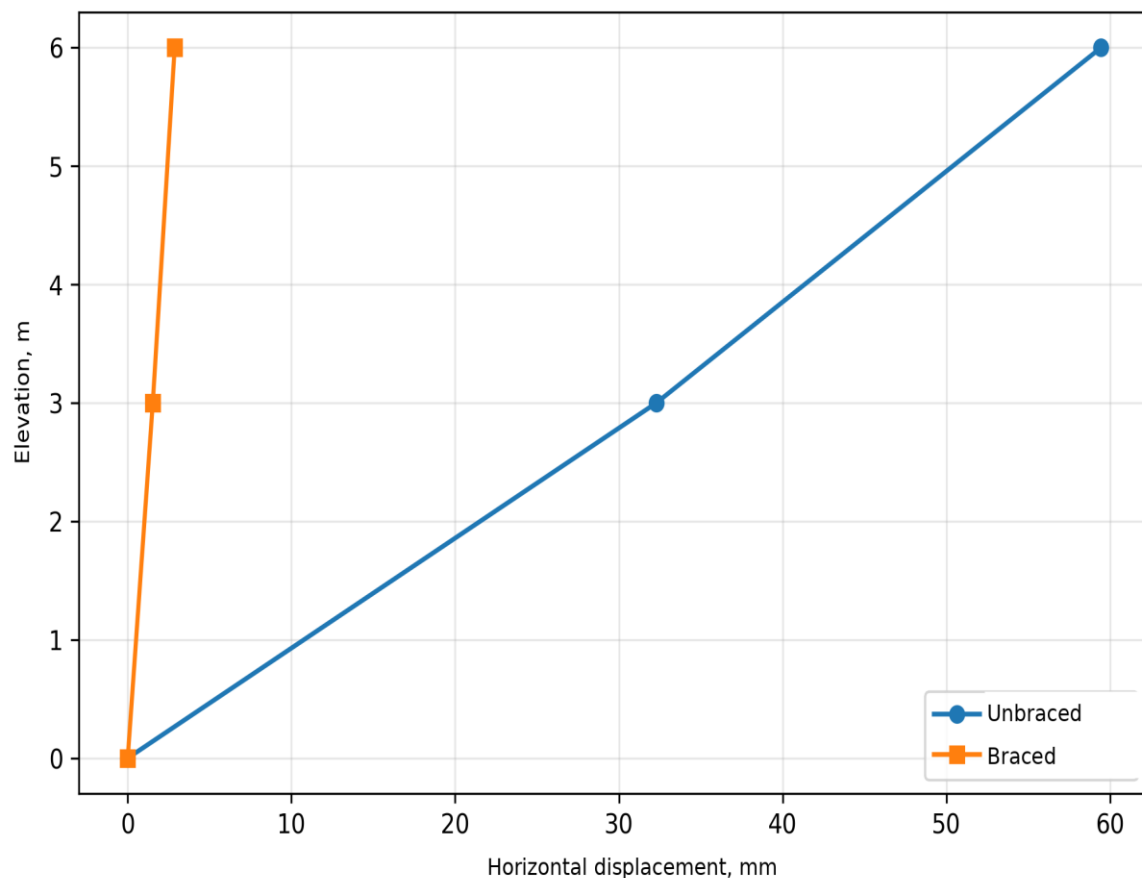


Figure 7 — Lateral displacement profiles of the braced and unbraced frames

The reduction in drift results from the transfer of most of the storey shear through axial action in the diagonals. In the braced configuration, beams and columns retain their roles in transferring vertical loads and completing the bay framework, but are not required to provide all lateral stiffness through bending. This separation is particularly

important for GFRP, whose low elastic modulus makes moment-frame action flexible and whose connections often do not attain the stiffness characteristic of a welded steel frame.

6.3. Sensitivity to joint rotational stiffness

Table 8 — Effect of k_0 on lateral response

k_0 , MN·m/rad	uroof, unbraced, mm	uroof, braced, mm	θ_1 , unbraced, ‰	θ_1 , braced, ‰
0	497.62	2.98	54.69	0.523
1	103.77	2.93	16.08	0.517
5	66.76	2.88	11.69	0.512
15	59.44	2.87	10.76	0.510
50	56.76	2.86	10.42	0.509
Rigid	55.60	2.86	10.26	0.509

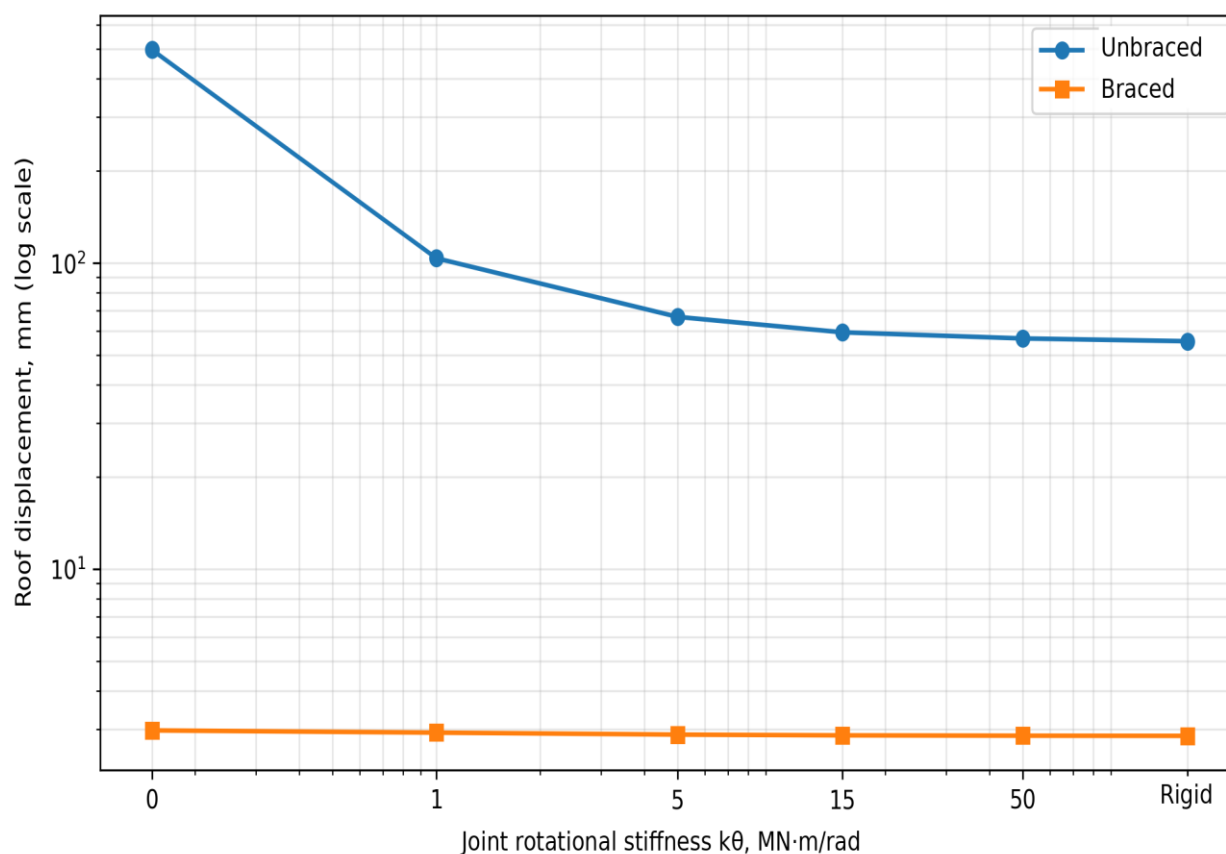


Figure 8 — Roof displacement as a function of joint rotational stiffness

For the unbraced frame, changing from pinned joints to $k_\theta = 1 \text{ MN}\cdot\text{m/rad}$ reduces roof displacement from 497.62 to 103.77 mm; increasing the stiffness further to 15 $\text{MN}\cdot\text{m/rad}$ gives 59.44 mm, while the rigid-joint idealisation gives 55.60 mm. The difference between pinned and rigid models is therefore almost ninefold. Even between $k_\theta = 1$ and 15 $\text{MN}\cdot\text{m/rad}$, the difference is 42.9%.

For the braced frame, the same range of k_θ changes displacement only from 2.98 to 2.86 mm. From $k_\theta = 5 \text{ MN}\cdot\text{m/rad}$ to the rigid-joint limit, the change is less than 1%. This is the key system-level result: reliably designed diagonals do not eliminate connection strength checks, but they make global drift insensitive to uncertainty in joint rotational stiffness. A braced configuration is therefore preferable for the first implementation of the hybrid building, rather than attempting to create a fully moment-resisting GFRP frame without experimentally verified rigid joints.

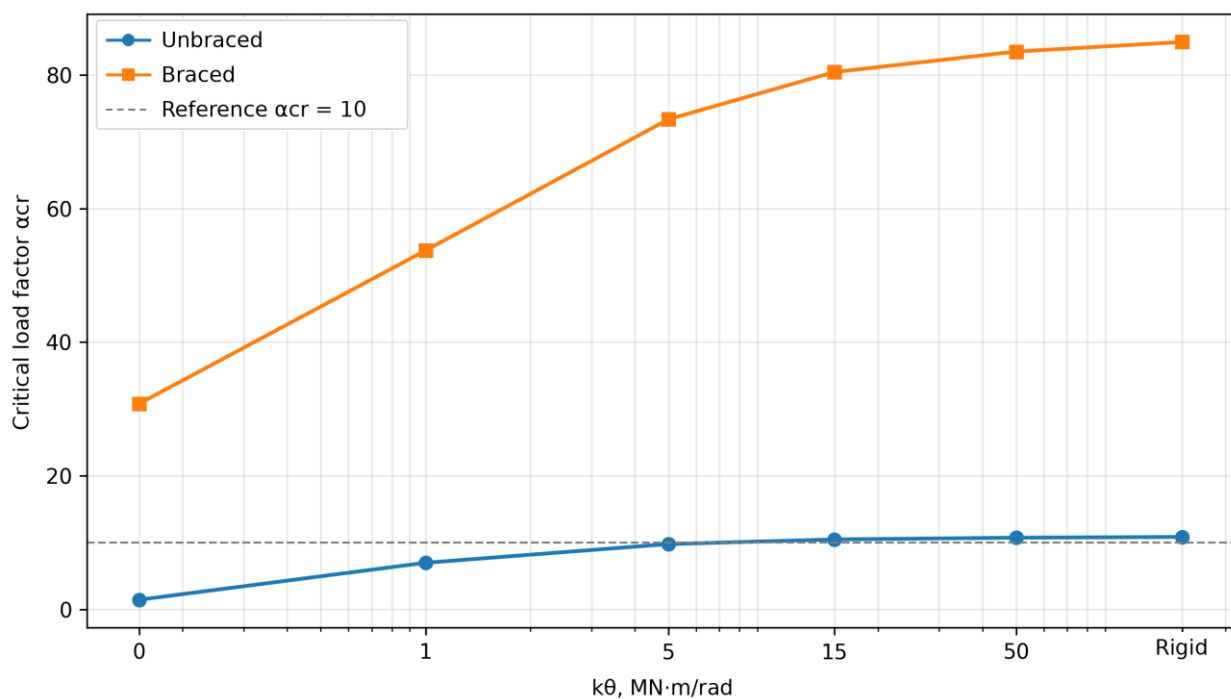
6.4. Elastic stability and second-order effects

Under the vertical load q_{ULS} , the critical load factor α_{cr} of the unbraced frame with pinned joints is 1.47. The corresponding approximate factor η reaches 3.14, indicating that linear analysis of this configuration is inadmissible. A stiffness of $k_\theta = 1 \text{ MN}\cdot\text{m/rad}$ already increases α_{cr} to 7.00, but second-order effects remain appreciable. At the baseline $k_\theta = 15 \text{ MN}\cdot\text{m/rad}$, $\alpha_{\text{cr}} = 10.49$ and $\eta = 1.105$.

The braced configuration gives $\alpha_{\text{cr}} = 30.79$ even with pinned joints and 80.41 at $k_\theta = 15 \text{ MN}\cdot\text{m/rad}$; the corresponding η values are 1.034 and 1.013. Thus, the diagonals not only reduce drift but also provide a substantial reserve of global elastic stability. However, α_{cr} pertains to an ideal linear-elastic model and does not account for local section-wall buckling, buckling of an individual diagonal, base flexibility or joint failure.

Table 9 — Critical load and second-order amplification factors

$k\theta$, MN·m/rad	α_{cr} , unbraced	η , unbraced	α_{cr} , braced	η , braced
0	1.47	3.139	30.79	1.034
1	7.00	1.167	53.72	1.019
5	9.80	1.114	73.35	1.014
15	10.49	1.105	80.41	1.013
50	10.75	1.103	83.47	1.012
Rigid	10.87	1.101	84.91	1.012

Figure 9 — Variation in critical load factor with $k\theta$

6.5. Dynamic stiffness

For the modal comparison, the masses at the two levels are taken as 15 and 13 t. In the unbraced baseline model, the first and second periods are 1.040 and 0.364 s. In the braced configuration, they decrease to 0.228 and 0.083 s.

The approximately fivefold reduction in the first period reflects increased lateral stiffness, not a universal advantage under earthquakes: changing the period moves the system along the response spectrum, so a final conclusion requires code-based spectral and nonlinear analysis for the site [36, 37].

Table 10 — Dynamic characteristics of the baseline model

Configuration	T1, s	T2, s	uroof under H, mm
Unbraced	1.040	0.364	59.44
Braced	0.228	0.083	2.87

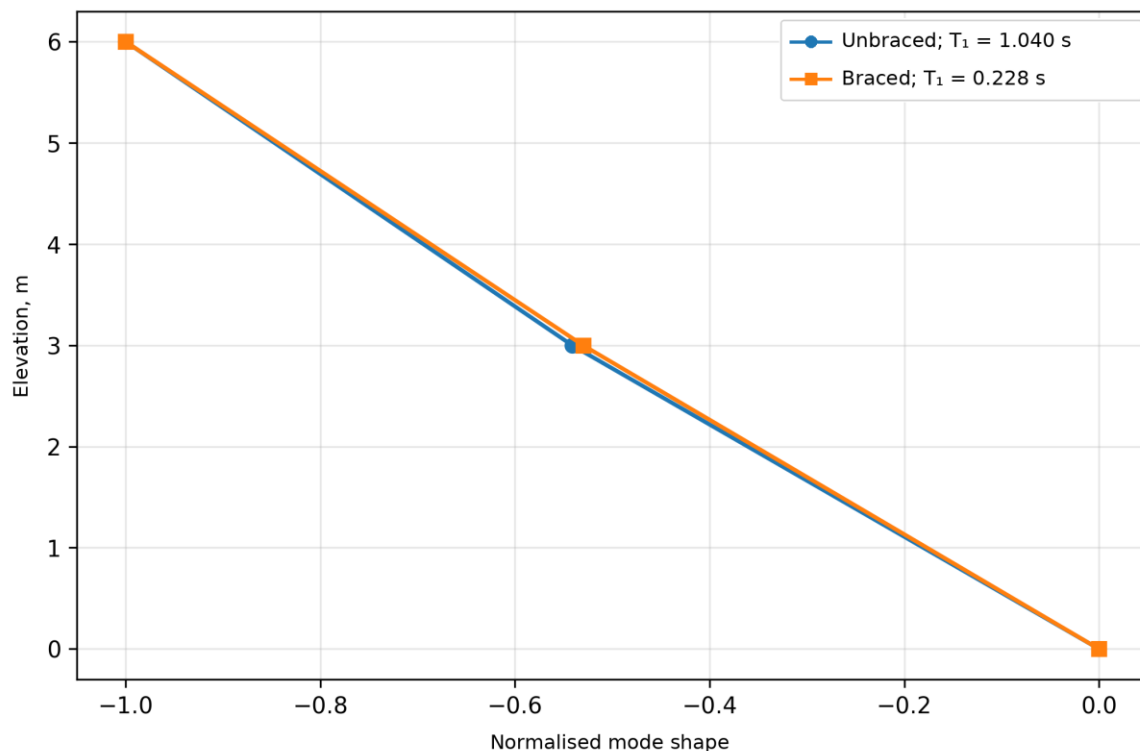


Figure 10 — Normalised first vibration modes of the frames at $k\theta = 15 \text{ MN}\cdot\text{m/rad}$

In the unbraced frame, the first period is also sensitive to $k\theta$: 2.912 s for pinned joints and 1.009 s for rigid connections. In the braced system, the range is only 0.232–0.228 s. Connection uncertainty can therefore substantially distort the calculated spectral forces of the unbraced system, whereas its influence on the period is small when bracing is present.

6.6. Internal forces and elastic stresses

For the V-ULS + H combination at $k\theta = 15 \text{ MN}\cdot\text{m/rad}$, the maximum column axial force is virtually unchanged

by the introduction of bracing: 135.50 kN without bracing and 134.43 kN with bracing. However, the flexural contribution of the columns decreases sharply. The maximum calculated normal stress from Eq. (12) decreases from 53.71 to 20.21 MPa. In the beams, the maximum moment decreases from 34.25 to 19.36 kN·m and the normal stress from 44.38 to 26.08 MPa. The braces carry up to 30.02 kN, corresponding to an elastic axial stress of 10.20 MPa.

Table 11 — Maximum internal forces and elastic stresses under the baseline combination

Quantity	Unbraced	Braced	Change
Roof displacement, mm	59.44	2.87	−95.2%
Column $N_{\max}$, kN	135.50	134.43	−0.8%
Beam $M_{\max}$, kN·m	34.25	19.36	−43.5%
Column $\sigma_{\max}$, MPa	53.71	20.21	−62.4%
Beam $\sigma_{\max}$, MPa	44.38	26.08	−41.2%
Beam $\tau_{\max}$, MPa	6.56	5.25	−19.9%
Brace $N_{\max}$, kN	—	30.02	—

Note. The stresses are calculated elastic values and are not compared with resistance values in the absence of verified material properties and service-condition factors.

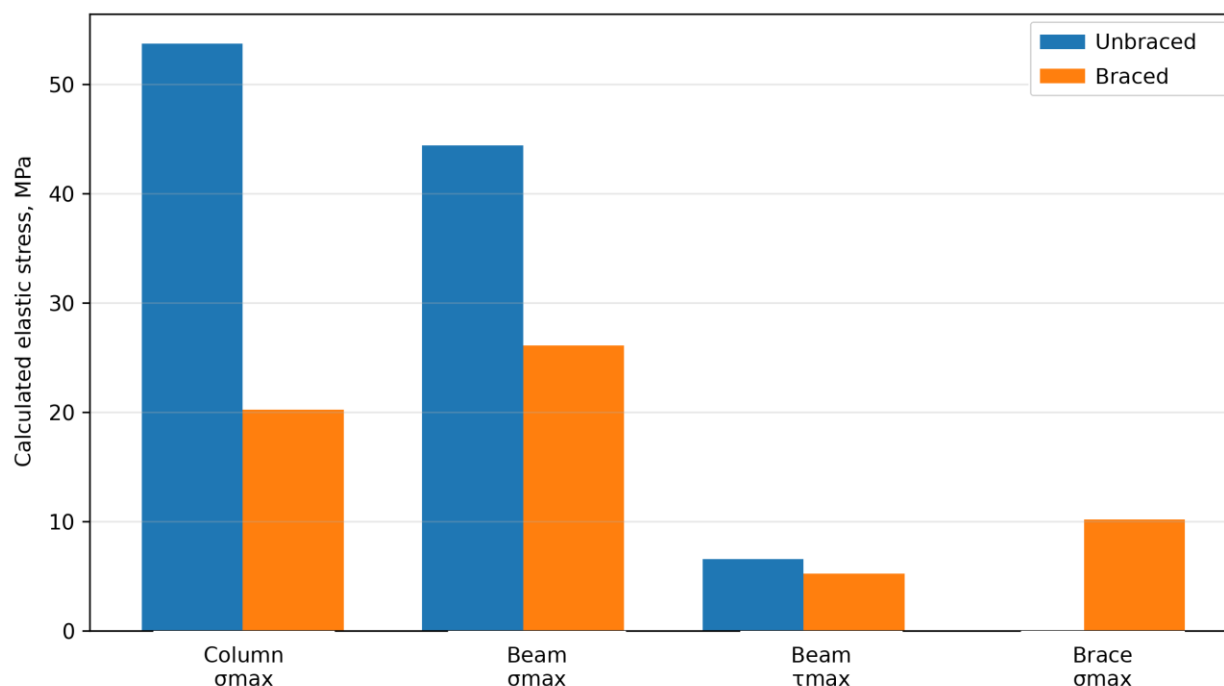


Figure 11 — Comparison of maximum elastic stresses in braced and unbraced frames

Despite the relatively low stresses, the design load-bearing capacity cannot be regarded as verified. The beam requires checks of local web and flange buckling, transverse shear, and bearing at supports and bolt groups. The column requires checks of axial force–bending interaction, local and global stability, initial curvature and weakening by holes. Braces require tension, compression and buckling checks, together with consideration of gusset-plate eccentricities.

6.7. Screening assessment of cross-sectional stability

The column radius of gyration is 76.91 mm and the geometric slenderness $L/r = 39.0$. The Euler load of 1346 kN is approximately ten times the maximum axial force under the baseline combination. This estimate explains the high α_{cr} of the braced frame but does not exclude a

local mode. For the hollow column, the clear wall-width-to-thickness ratio is $176/12 = 14.7$; for the beam web, it is $276/12 = 23.0$.

A simplified isotropic-equivalent plate formula with $ET = 8$ GPa and $k = 4$ is used as a preliminary, non-code screening method. The resulting elastic local buckling stresses are 130.5 MPa for the column wall, 53.1 MPa for the beam web and 254.6 MPa for the beam flange. The lowest value exceeds the calculated elastic stresses, but this check does not account for the orthotropic wall structure, corner radii, interaction between adjacent plates or imperfections. The final decision must be based on the formulations of CEN/TS 19101 or ASCE/SEI 74-23 and/or a nonlinear shell model verified by testing [1–4, 8, 12, 13].

$$\sigma_{cr,pl} \approx k \pi^2 ET / [12(1 - \nu^2)] \cdot (t/b)^2 \quad (13)$$

Table 12 — Screening indicators of stability

Quantity	Column	Beam	Comment
L/r	39.0	—	Column length: 3.0 m
N _{cr,E}	1346 kN	—	Global mode, K = 1.0
Wall b/t	14.7	23.0	Clear width between corners
Wall $\sigma_{cr,pl}$	130.5 MPa	53.1 MPa	Isotropic-equivalent estimate
Maximum elastic σ	20.2 MPa	26.1 MPa	Braced baseline model

Note. This table is not a load-bearing capacity verification and is used only to rank potential failure modes.

7. Parametric analysis

7.1. Sensitivity to EL and GLT

The longitudinal modulus is varied from 18 to 28 GPa and the shear modulus from 2 to 4 GPa. For the beam with $k\theta = 15 \text{ MN}\cdot\text{m/rad}$, deflection ranges from 4.92 mm

at EL = 18 GPa and GLT = 2 GPa to 3.18 mm at EL = 28 GPa and GLT = 4 GPa. The baseline values of 23 and 3 GPa give 3.82 mm. Changing EL independently has a greater effect on the bending component, whereas GLT controls the constant shear contribution.

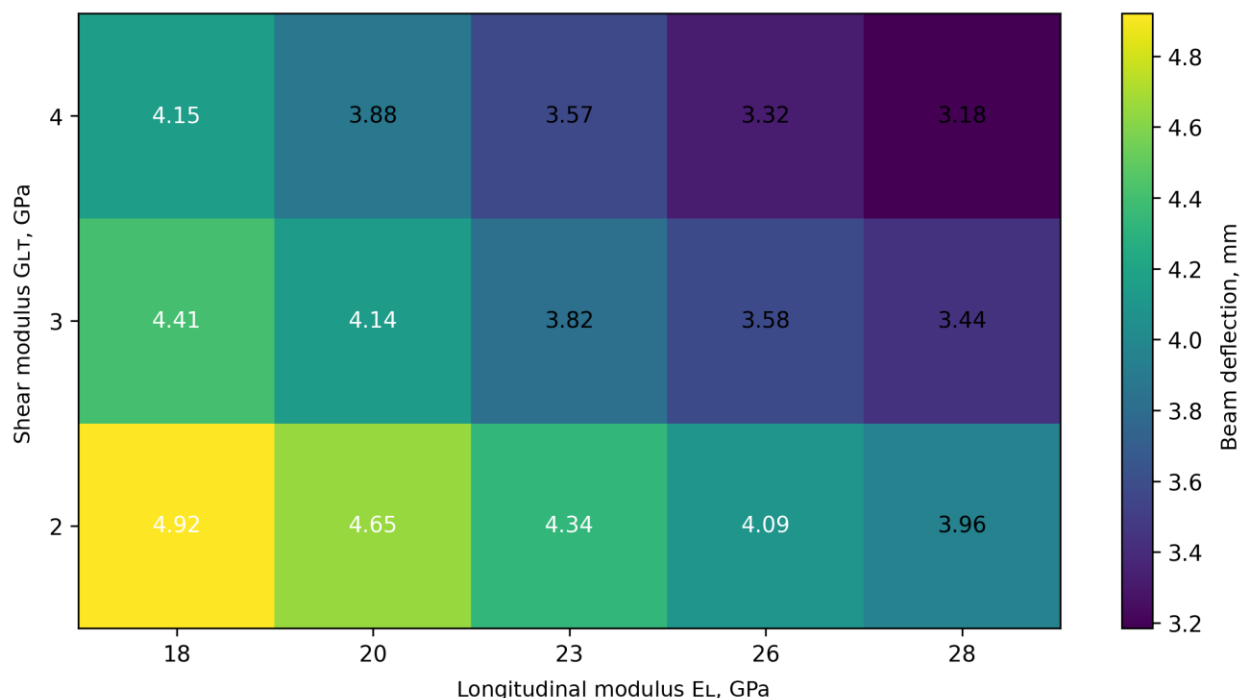


Figure 12 — Sensitivity of beam deflection to longitudinal and shear moduli

For the braced frame, lateral displacement is governed primarily by the axial stiffness of the diagonals and EL, whereas the influence of GLT is limited to beam and column bending. This does not make GLT a secondary parameter: it remains critical to vertical deflection, short members, local connection regions and stability calculations. The testing programme must therefore include direct determination of the shear modulus, not longitudinal tensile testing alone.

7.2. Sensitivity to brace area

The diagonal area is varied from $0.5A_0$ to $1.5A_0$ with unchanged geometry. Halving the area increases roof displacement from 2.87 to 4.79 mm and the first period from 0.228 to 0.297 s. Increasing the area by 50% reduces displacement to 2.20 mm and the period to 0.198 s. The factor α_{cr} changes moderately, from 76.76 to 83.45, because once a stiff triangulated configuration is established, its value is also limited by the columns and boundary conditions.

Table 13 — Effect of diagonal brace area

A/A ₀	A, mm ²	u _{roof} , mm	T ₁ , s	α _{cr}
0.50	1472	4.79	0.297	76.76
0.75	2208	3.52	0.254	78.68
1.00	2944	2.87	0.228	80.41
1.25	3680	2.47	0.211	81.99
1.50	4416	2.20	0.198	83.45

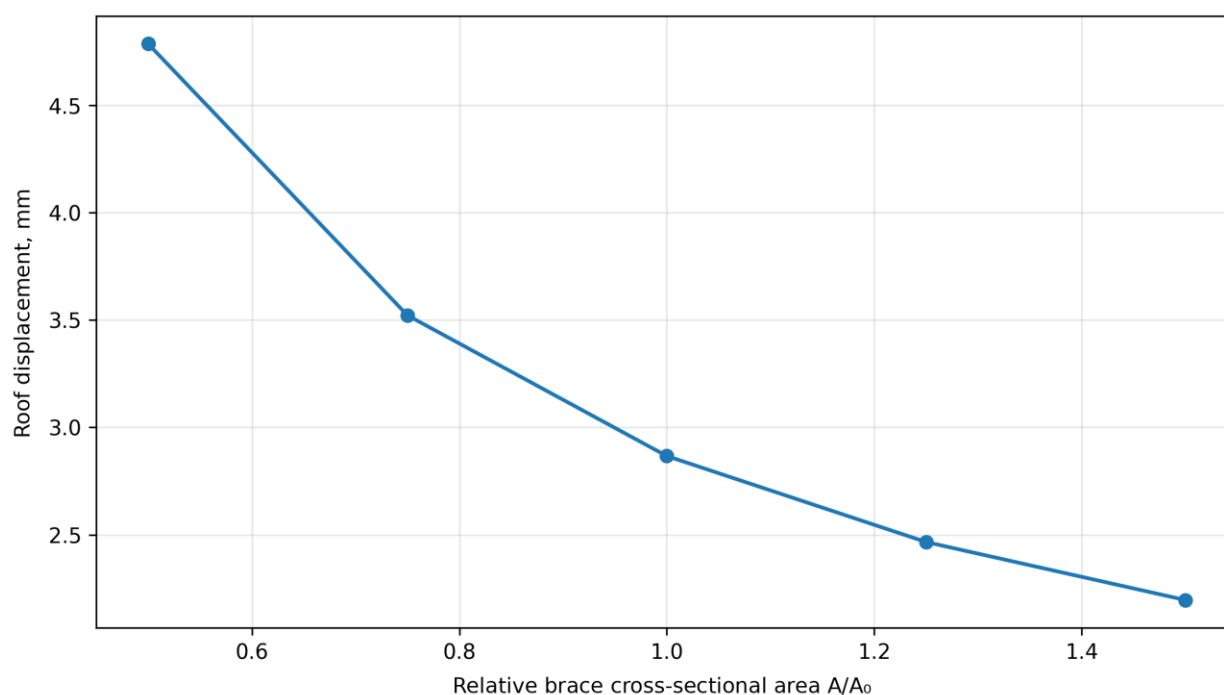


Figure 13 — Roof displacement as diagonal brace area varies

Brace selection must not be based on global displacement alone. Checks are required for compression-diagonal buckling, net-section tension, gusset plates, bolts, eccentricity and cyclic behaviour. In a cross-braced configuration, each diagonal may be designed primarily for tension with nonlinear deactivation of the compression member, but displacements and internal forces must then be recalculated for unilateral action.

8. Discussion

The results confirm that serviceability and system stability can govern the design of a pultruded frame before material strength does. In the unbraced configuration, even idealised rigid joints do not provide low drift: roof displacement is 55.60 mm under the selected force pattern. This is attributable to the relatively

low EL of the columns and beams. Attempting to resolve the problem solely by increasing profile thickness increases mass and cost but does not address connection uncertainty.

The bracing subsystem is a more effective means of controlling deformation. It reduces not only lateral displacements but also bending stresses, sensitivity to $k\theta$, the first period and second-order effects. Beam-to-column joints can consequently be treated as connections whose principal task is to transfer vertical shear and a limited moment reliably, rather than to create a fully moment-resisting frame. This simplifies experimental qualification of the first version of the system.

Bracing must not, however, be interpreted as a universal solution. Concentrating lateral forces in the outer bays increases the demands on gusset plates, anchors and

foundations. Architectural openings may preclude X-diagonals; alternatives then include portal bracing frames, rigid diaphragms, composite panels or steel/hybrid cores. Any alternative must be represented explicitly in the model rather than replaced by uncertain action of the printed wall.

A separate finding concerns transverse shear. With semi-rigid and rigid ends, the bending component of deflection decreases while the shear component remains, reaching 27–33% of the total. Increasing $k\theta$ without refining GLT therefore creates a false impression of accuracy. In

practice, bolt-hole deformation, cleat shear and profile-wall flexibility are also important; they may be included in a generalised $k\theta$ or represented by separate springs.

Comparison with the contemporary literature is qualitatively consistent with the observed semi-rigidity of connections and the susceptibility of GFRP members to deformation- and stability-related limits [9, 16–29]. Quantitative values from the present model must not be transferred to another profile: changes in fibre architecture, resin, corner radius and connection type can substantially alter EL, GLT, local stability and $k\theta$.

Table 14 — Practical implications for design

Design situation	Recommended model	Response to be checked
First building implementation	Independent bracing system; non-load-bearing walls	Drift and forces in braces and anchors
Untested joint	Range of $k\theta$, including a lower-bound estimate	Moment and displacement envelopes
Deep GFRP beam	Timoshenko element or 3D model	Total and long-term deflection
Column	Global and local stability	N–M interaction and imperfections
X-bracing	Tension/compression or tension-only	Cyclic behaviour and stability
3D-printed wall	Excluded from global stiffness	Out-of-plane pressure and movement clearances
Seismic analysis	Site-specific modal/nonlinear model	Spectral response, drift and joint damage

9. Programme of experimental and numerical verification

A material–member–joint–frame–building sequence is proposed for progressing from the demonstrative model to a design model. Each level must provide parameters for the next, not merely an ultimate load.

1. Profile material: longitudinal and transverse tension, compression, shear, bending, density, glass-transition temperature and water absorption; determination of mean and characteristic values for the actual batch.
2. Full-size beam: four-point bending with measurement of deflection, rotations and strains; separate identification of EL and GLT; sustained-load testing.
3. Full-size column: short, intermediate and slender specimens with measurement of initial curvature and local buckling; digital image correlation.
4. Beam-to-column joint: monotonic and cyclic moment–rotation curves, initial clearance, slip, secant stiffness, residual rotation and failure mode.
5. Bracing connection: tension and compression, eccentricity, bolt-group action, bearing, splitting and gusset-plate buckling.
6. Portal frame: lateral loading with simultaneous vertical force; verification of internal-force distribution and $P-\Delta$ effects; comparison of displacement fields with the model.
7. Frame–printed-wall assembly: cyclic interstorey drift under out-of-plane pressure, with monitoring of sliding anchors, joints and residual clearance.
8. Full-scale building: monitoring of displacements, temperature, humidity, vibrations and connection condition over at least one seasonal cycle.

Table 15 — Minimum parameters to be identified

Level	Model parameter	Acceptance criterion
Coupon	EL, ET, GLT, ν and strengths	Statistically evaluated properties
Beam	EI, $\kappa G A \nu$ and load-duration factor	Agreement of load–deflection curves
Column	Initial curvature, local and global modes	Reproduction of critical load and mode
Joint	$k\theta(M)$, slip and ultimate moment	Model error within the working range $\leq 10\text{--}15\%$
Brace	EA, compression-diagonal behaviour and gusset flexibility	Agreement of force and displacement
Frame	Drift, moment distribution and α_{cr}	Reproduction of the experimental envelope
Building	Periods and service displacements	Agreement between monitoring and the digital model

Note. The numerical criterion of 10–15% is a research target to be refined by the testing programme and the requirements of the journal or code-based procedure.

10. Limitations of the study

The model is two-dimensional and does not account for building torsion, three-dimensional diaphragm action, foundation flexibility or non-uniform distribution of storey forces. The diagonals act linearly in tension and compression; buckling of the compression diagonal is not modelled. The material is assumed linear elastic, without damage, creep or temperature- and moisture-induced degradation. Each joint is represented by a single linear rotational spring without initial slip, nonlinearity or failure.

Geometric stability is investigated through a linear eigenvalue problem for the ideal geometry. Design assessment requires geometrically and materially nonlinear analysis incorporating initial imperfections, local shell modes and actual joint response curves. The screening plate formula in Eq. (13) is not a code-based verification of an orthotropic hollow profile.

The adopted loads and masses are not associated with a specific construction address. Seismic design in Uzbekistan requires the assignment of site seismicity, ground category, spectrum, behaviour factors and combinations in accordance with the applicable codes [36, 37]. The resulting periods and drift ratios are used only to compare configurations of the same model.

11. Conclusions

1. A parametric finite element model of a two-storey pultruded GFRP frame has been developed, incorporating member transverse shear, joint rotational flexibility, diagonal bracing, elastic stability and modal characteristics.

2. For a $300 \times 150 \times 12$ mm beam spanning 3.6 m under $q = 12.6$ kN/m with $k\theta = 15$ MN·m/rad, the support moment is 12.38 kN·m, the span moment is 8.03 kN·m and the deflection is 3.82 mm. The shear contribution is 1.03 mm, or 26.9% of the total.
3. Roof displacement in the unbraced frame is strongly dependent on $k\theta$, ranging from 497.62 mm for pinned joints to 55.60 mm for rigid joints. At the baseline $k\theta = 15$ MN·m/rad, it is 59.44 mm and the maximum drift ratio is 10.76 %.
4. Introducing X-bracing reduces roof displacement to 2.87 mm and the maximum drift ratio to 0.510 %. In the braced configuration, varying $k\theta$ from 5 MN·m/rad to the rigid-joint limit changes displacement by less than 1%.
5. At $k\theta = 15$ MN·m/rad, bracing increases the critical load factor from 10.49 to 80.41 and reduces the approximate second-order factor from 1.105 to 1.013.
6. The first period of the baseline model decreases from 1.040 s without bracing to 0.228 s with bracing. Uncertainty in $k\theta$ substantially affects the period of the unbraced system and has almost no effect on the braced system.
7. Under combined vertical and lateral loading, bracing reduces the maximum elastic column stress from 53.71 to 20.21 MPa and the beam stress from 44.38 to 26.08 MPa; the maximum diagonal force is 30.02 kN.
8. Global stability and low drift must be provided independently of the 3D-printed walls. For the first

implementation of the system, a braced frame with experimentally determined joint response curves is recommended; the printed walls should remain non-load-bearing and deformation-decoupled.

9. Before final resistance values are published and the system is used in design, verification is required at the material, member, joint, frame and frame-wall assembly levels, including sustained, cyclic, thermal and seismic actions.

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