Experimental and numerical analysis of sandwich metal panels

Zbigniew Pozorski, Monika Chuda-Kowalska, Robert Studziński, Andrzej Garstecki

Poznan University of Technology, Institute of Structural Engineering, Poznan, Poland

OUTLINE

Aim of the study
Experimental determination of mechanical parameters
Local buckling (wrinkling)
Failure maps
Optimization of sandwich panels
AIM OF THE STUDY

1) Proper description of structural behaviour of sandwich panels (non-homogeneous, anisotropic core, profiling of metal faces, non-uniform boundary conditions)

2) Precise analysis and prediction of failure mechanisms

3) Optimal design (minimum cost, maximum span, maximum load capacity)
EXPERIMENTAL DETERMINATION OF MECHANICAL PARAMETERS

Reliable methods of determination modulus $G_C$ of the core are still expected by producers and designers:

- $G_C$ strongly influences the response of the panel
- methods suggested by code 14509 provide results depending on the method and specimen’s scale

Tasks:
- verification of methods of identification the Kirchhoff modulus $G_C$ proposed by code 14509
- improvement of identification methods
**CLASSICAL METHODS**

1. Tests on short/long panels \( (w) \)

\[
W = W_B + W_S , \quad \text{(1)}
\]

\[
W_B = \frac{23 \cdot \Delta F \cdot L^3}{1296 \cdot B_S} , \quad \text{(2)}
\]

\[
G_C = \frac{\Delta F \cdot L}{6 \cdot B \cdot d_C \cdot \Delta w_S} . \quad \text{(3)}
\]

where:

- \( W_B, W_S \) – bending and shear deflection
- \( L, B \) – span and width of the panel
- \( d_C \) – depth of the core
- \( B_S \) – flexural rigidity
NON-CLASSICAL METHODS

1. Measurement of angles of rotation ($\gamma$)
Assessment of the shear modulus

where:
\( \gamma_{01}, \gamma_{02} \) - total slope of deflection line
\( \alpha_0 \) - the angle of rotation of the cross-section
\( \gamma \) - the angle of rotation of the core

\[ \gamma_0 = 0,5 \cdot (\gamma_{01} + \gamma_{02}), \quad (4) \]
\[ \gamma = \gamma_0 - \alpha_0, \quad (5) \]

\[ G_C = \frac{V}{\gamma \cdot B \cdot d_C} \quad (6) \]
where:
\( V \) - the shear force
2. Shear test ($\gamma$)

The force $F$ acts on the middle plate and the vertical displacement $w$ of the rigid plate is measured.

\[
\gamma = \frac{w}{d_c}, \quad (7)
\]

\[
G_c = \frac{F}{2 \cdot \gamma \cdot B \cdot L}. \quad (8)
\]

1 – rigid steel plate (10x240x550mm)
2 – sandwich panel (100x240x500mm)
3. Torsion test ($\phi$)

\[
\phi' = \frac{M_s}{G_C \cdot I_0},
\]

(9)

\[
\phi' = \frac{\varphi}{L},
\]

(10)

\[
I_0 = \frac{\pi \cdot D_1^4}{32}.
\]

(11)

where:

$M_s$ – torsion moment

$\varphi$ – angle of specimen rotation

$L$ – length of the sample

$D_1$ – diameter of the cylinder cross-section

$I_0$ – central second order moment of the area of the cylinder cross-section
Discussion of the results

Experimental results of $G_C$ from methods proposed by code 14509

<table>
<thead>
<tr>
<th>Type of sample</th>
<th>Width of the sample</th>
<th>$G_C$ [MPa]</th>
<th>$\delta$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short panel L=0.6m</td>
<td>B=0.1m</td>
<td>3.33</td>
<td></td>
</tr>
<tr>
<td>Long panel L=4.9m</td>
<td>Total width</td>
<td>4.87</td>
<td>46.2</td>
</tr>
</tbody>
</table>

Experimental results of $G_C$ from different bending methods

<table>
<thead>
<tr>
<th>Type of panel</th>
<th>Method of identification</th>
<th>$G_C$ [MPa]</th>
<th>$\delta$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal edge profiling exists</td>
<td>Measured $w$</td>
<td>4.87</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Measured $\gamma$</td>
<td>4.70</td>
<td>3.6</td>
</tr>
<tr>
<td>Longitudinal edge profiling cut off</td>
<td>Measured $w$</td>
<td>3.73</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Measured $\gamma$</td>
<td>3.59</td>
<td>3.9</td>
</tr>
</tbody>
</table>

Influence of longitudinal edge profiling on test results

- 30.6
- 30.9
LOCAL BUCKLING (WRINKLING)

Tasks:
- numerical modelling of structural response representing global and local effects
- analysis of progressive damage, contact effects, wrinkling
- assessment of the influence of discontinuity of the core
LABORATORY TESTS

- supporting system
- load cell HBM C 6A 200kN 0.5 class
- displacement transducers HBM WA L 100mm
- tensometers HBM LY 10 mm
- HBM catman 4.5 software
NUMERICAL SIMULATIONS

1. Class of problems

Core: PU (isotropic, homogeneous, continuous)

Mineral wool (anisotropic, macro-homogeneous, non-continuous)

Profiling of faces: flat, micro-profiled, deep-profiled

Boundary conditions: transversal loads, thermal actions
2. Bending of flat (or micro-profiled, deep-profiled) panels

Material parameters:

**Faces:**  
\( E_F = 210 \text{ GPa}, \quad \nu_F = 0.3, \quad f_y = 270 \text{ MPa} \)  
\( \text{(elastic, ideal plastic)} \)

**Core (MW):**  
\( E_C = 12 \text{ MPa}, \quad \nu_C = 0.05, \quad G_C = 5.714 \text{ MPa} \)  
\( \text{(isotropic elastic)} \)

- \( E_t = E_3 = 14 \text{ MPa}, \quad E_p = E_1 = E_2 = 3 \text{ MPa} \)  
  \( \text{(ortotrophy)} \)
- \( G_p = G_{12} = 1.43 \text{ MPa}, \quad G_t = G_{13} = G_{23} = 6.36 \text{ MPa} \)
- \( \nu_{pt} = \nu_{13} = \nu_{23} = 0.03, \quad \nu_{tp} = \nu_{31} = \nu_{32} = 0.14 \)

**Interface:**  
\( K_{nn} = 12 \text{ MPa}, \quad K_{ss} = K_{tt} = 6 \text{ MPa} \)  
\( \text{(cohesive)} \)
Damage modelling in the interface layer

• Elasticity (uncoupled) for cohesive elements
\[
\begin{bmatrix}
    t_n \\
    t_s \\
    t_t
\end{bmatrix} =
\begin{bmatrix}
    K_{nn} & 0 & 0 \\
    0 & K_{ss} & 0 \\
    0 & 0 & K_{tt}
\end{bmatrix}
\begin{bmatrix}
    \varepsilon_n \\
    \varepsilon_s \\
    \varepsilon_t
\end{bmatrix}
\]

\( t_n \) – normal traction (stress)
\( t_s, t_t \) – shear tractions
\( \varepsilon \) - corresponding nominal strain

• Damage initiation
  - quadratic nominal stress
\[
\left\{ \frac{t_n}{t_n^o} \right\}^2 + \left\{ \frac{t_s}{t_s^o} \right\}^2 + \left\{ \frac{t_t}{t_t^o} \right\}^2 = 1
\]

\( t_n^o = 140 \text{kPa} \)
\( t_s^o = 100 \text{kPa} \)
\( t_t^o = 100 \text{kPa} \)

• Damage evolution
  Type: Displacement, Softening: Linear

• Contact in the core (between lamellas)
  - hard contact
  - exponential
  - Coulomb friction model
Example 1. Core: isotropic, continuous

\( q = 2.205 \text{ kN/m}^2 \)

faces: \( \sigma_{11} = -234.2 \text{ / } +102.7 \text{ MPa} \)

deflection: \( u = 2.954 \text{ cm} \)
Example 2. Core (mineral wool): orthotropic, non-continuous, "hard" contact

q = 0.521 kN/m²
faces: \( \sigma_{11} = -34.24 \, / \, +22.81 \) MPa
deflection: \( u = 0.6037 \) cm
Example 2a. Core: orthotropic, non-continuous

CSLIP – tangential motion of the surfaces during contact
Example 3. Non-uniform loading, continuous core

q = 5.285 kN/m²
faces: \( \sigma_{11} = -122.7 / +54.40 \) MPa
deflection: \( u = 1.659 \) cm

Discussion of the results
FAILURE MAPS

- Failure maps specify the failure modes for different combinations of the design parameters.

- Groups of interrelations:
  - thickness of the facings $t_{Fi}$ vs thickness of the core $d$,
  - shear modulus $G_C$ vs $d$,
  - shear modulus $G_C$ vs stiffness of the external ($k_1$) or internal ($k_2$) support.

- The isolines: the maximal allowable load $p(x)$ or temperature difference $\Delta T = T_2 - T_1$ for various combinations of the analysed parameters.
Failure maps – examples

- Mechanical load $p(x) = p$ [kN/m]
- Isolines of $p$ for variable $G_C$ and stiffness of the external (a) and internal (b) support

Comments:
- the response of structure with varied internal support stiffness is different from that with varied external support stiffness
- apparently the limits of the failure modes converge with the maximal capacity of the panel
- the general optimal capacity level is nearly the same and the sensitivity of the capacity of the sandwich panel is much less than in the case of the thermal actions
Failure maps – examples

- Thermal load $\Delta T$ [°C]
- Isolines of $\Delta T$ for variable $G_C$ and stiffness of the external (a) and internal (b) support

Comments:
- large sensitivity of the panel capacity along the limits of the failure modes
- elastic supports leads to significant improvement of the capacity of the sandwich panel (40% - 60%)
- quite opposite results are observed in the case of mechanically loaded panels
OPTIMIZATION
Motivation for optimal design

- core failure
- facing failure
- deflection
- minimal variance in types of panels
- maximal range of application
- minimum cost

ULS and SLS
Market demands
Optimization – Problems formulation

I. Optimization of the material and geometrical parameters

- design vector: \( s = [t_1, t_2, D, G_C, L] \)
- two criterion fitness function:
  - minimum cost \( F_C(s) \)
  - maximum length of the span \( L \)
- constraints:
  - ULS and SLS
  - box conditions

II. Optimization of the support conditions

- design vector: \( s = [k_j, \delta_j, L], j=1,.., n \) where \( j \) is the support number
- two criterion fitness function:
  - maximum load multiplier \( \lambda \)
  - maximum span \( L \)
- constraints:
  - ULS and SLS
  - box conditions

- The problems are nonconvex hence Distributed Parallel Evolutionary Algorithm was used
Optimization – Example

Influence of temperature and load intensity on cost function

Cost [$1\cdot m^{-1}$] vs. $L^{-1}[m^{-1}]$ and $L [m]$ for different temperatures and load intensities.

- $q = 0.75 \text{kN/m}$
- $q = 1.00 \text{kN/m}$
- $q = 2.00 \text{kN/m}$
- $\Delta T = -40^\circ C$
- $\Delta T = -65^\circ C$
- $\Delta T = -90^\circ C$
Optimization – Example

- Elastic support with a gap, thermal action $\Delta T$ [°C]

Design parameters:
- $k_1$ - stiffness of the external support
- $\delta_2^-$ - gap in the internal support
- $L$ - span length

Measure of the quality of the structure:
- $\lambda$ - load multiplier of the thermal load $\Delta T$,
  where $\lambda = 1$ means maximal capacity of the thermally loaded panel on rigid supports
PART 2
A. Garstecki

Research activity in Steel Structures

Prof. M. Szumigała, Dr. K. Rzeszut, Dipl.Eng. M. Chybiński

(Division of Metal Structures)
1. Statically and stability analysis of lattice thin-walled structures

2. Composite steel-concrete beams
Motivation

- Wide implementation in civil engineering
- Nonlinear behaviour due to deformation of initial contour and geometrical imperfections

Tasks

- Buckling and postbuckling analysis of columns and beams made of single Σ and double Σ cross-section
- Sensitivity to imperfections and clearances
- Optimal configuration of ribs in I beams
- Moment-curvature relation and limit curves for steel-concrete composite beams
1. K. Rzeszut, Nonlinear stability analyses of thin-walled structures

Equilibrium equations:

\[ F(P, U) = 0, \quad (1) \]

where \( F \) is a nonlinear differential operator, \( P \) is a load vector and \( U \) denotes a displacement vector. It can be written in incremental form

\[ (K^O + K^\sigma + K^U)\Delta U = \Delta P, \quad (2) \]

where:

\( K^O \) - small-displacement stiffness matrix

\( K^\sigma \) - initial stress matrix

\( K^U \) - load stiffness matrix.

Eq. 3 accounts for all nonlinearities of the problem.

Riks method is used to solve eq. 3 for both stable and unstable postbuckling behaviour.
GEOMETRIC IMPERFECTIONS
Postbuckling analysis by introducing a geometric imperfection pattern to the “perfect” geometry.

Lowest buckling modes are scaled and added to the perfect geometry to create the perturbed mesh.

Imperfections have the form:

\[
\mathbf{U} = \frac{1}{\lambda} \mathbf{K}^G \mathbf{U} = 0 \quad (3)
\]

where:

\[
\mathbf{u} \equiv \tilde{\mathbf{u}} = \begin{bmatrix} \tilde{u}_r \end{bmatrix} = \sum_{i=1}^{n} \alpha_i \mathbf{U}_i \quad (4)
\]

\( \mathbf{U}_i \) - the buckling mode,
\( \alpha_i \) - the associated scale factor,
\( n \) - the number of eigenmodes used to create the perturbed mesh.
Initial imperfections and clearances in stability analysis

Asymmetrical, unstable bifurcation point

a) \( \eta = 0.2 \),

\[
\frac{P}{P_{cr}}
\]

b) \( \eta = 1 \)

\[
\frac{P}{P_{cr}}
\]

\[\bar{\Delta} = \Delta / L\]

Where \( \eta = k_1 / k_2 \) - non-dimensional stiffness coefficient
Connectors of cold formed double SIGMA elements

“Link” - preserves only the constant distance between points A and B.

“Tie” - the rotations are constrained.

“Slot” - accounts for the clearances in the connection.
Different spacing \( l_1 \) (\( l_1=0.5\text{m}, \ l_1=1.0\text{m}, \ l_1={l}/2, \ l_1={l} \) and \( l_1=0 \)) and different numerical models of connectors was analysed.
Non-linear stability analysis of a double $\Sigma$ beam

Load proportionality factor of column made of double $\Sigma$ cross section for global ($g$), local ($l$) shapes of imperfections
Stability of laterally braced purlins

Fig 3. Shapes of buckling mode for purlins:
  a) without bracings, b) with two bracings.
2. M. Chybiński, Optimal configuration of ribs in welded girders
3. M. Szumigała, Composite steel-concrete elements
Limit curves M-N for various numerical models
Conclusions

• Small clearances influence the stability response in a similar way as small imperfections.

• Thick head plates in girders improve torsional stiffness, however usual habits are too conservative. Thick head plates decrease rotational capacity of bolted joint.

• Diagonal ribs in welded girders are definitely better than orthogonal ones (in linear and non-linear regimes).

• In steal concrete composite structures the normal force drastically influences the moment-curvature relation.
Last information on the activity

Students’ MSc diploma works made for Rautaruukki Corporation in 2008 and 2010.

Thank you for your attention