Metal shear panels for seismic protection of framed structures

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METAL SHEAR PANELS

Stiffened shear plate

Adopted metals

Tension field mechanism (elastic buckling)

Pure shear mechanism (no-buckling)
SHEAR PANELS ARRANGEMENTS IN THE FRAMEWORK

Compared to full bay type panels, bracing type and pillar type panels allow a significant dissipative behaviour already for low interstorey drift values.
ALUMINIUM ALLOY USED FOR SHEAR PANELS—Monotonic behaviour

Cycles of heat treatment of the aluminium alloy

<table>
<thead>
<tr>
<th>N° PHASE</th>
<th>TEMPERATURE [°C]</th>
<th>EXPOSURE TIME [hours]</th>
<th>BRINNELL INDEX [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>iniziale</td>
<td>ambiente</td>
<td>/</td>
<td>69</td>
</tr>
<tr>
<td>1</td>
<td>150</td>
<td>4</td>
<td>68</td>
</tr>
<tr>
<td>2</td>
<td>230</td>
<td>4</td>
<td>67</td>
</tr>
<tr>
<td>3</td>
<td>280</td>
<td>4</td>
<td>44</td>
</tr>
<tr>
<td>4</td>
<td>330</td>
<td>4</td>
<td>35</td>
</tr>
</tbody>
</table>

Material | $f_{0.2}$ [N/mm²] | $f_u$ [N/mm²] | $e_u$ % | $E$ [N/mm²] | $E/f_{0.2}$ [N/mm²] | $a = f_u/f_{0.2}$ |
<table>
<thead>
<tr>
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</tr>
</thead>
<tbody>
<tr>
<td>LYS steel</td>
<td>86</td>
<td>254</td>
<td>50</td>
<td>210000</td>
<td>2442</td>
<td>2.95</td>
</tr>
<tr>
<td>Nominal Pure Aluminium (EN-AW 1099A)</td>
<td>15-20</td>
<td>40-50</td>
<td>40-50</td>
<td>70000</td>
<td>3500-4666</td>
<td>2-3.3</td>
</tr>
<tr>
<td>Nominal Pure Aluminium (EN-AW 1050A)</td>
<td>30-70</td>
<td>70-100</td>
<td>20-40</td>
<td>70000</td>
<td>1000/2333</td>
<td>2.33-3.33</td>
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<tr>
<td>Employed Pure Aluminium (EN-AW 1050A)</td>
<td>80</td>
<td>100</td>
<td>5</td>
<td>70000</td>
<td>875</td>
<td>1.25</td>
</tr>
<tr>
<td>Heat Treated Pure Aluminium (EN-AW 1050A)</td>
<td>21.3</td>
<td>80</td>
<td>45</td>
<td>70000</td>
<td>3286</td>
<td>3.76</td>
</tr>
</tbody>
</table>

Not heat treated

Heat treated

$\sigma_{0.2} = 115$ MPa

$\sigma_u = 69$ MPa

$\sigma_{0.2} = 20$ MPa

a) failed not heat-treated specimen

AW 1050A specimen

b) failed heat-treated specimen
ALUMINIUM ALLOY USED FOR SHEAR PANELS - Cyclic behaviour

BUCKLING INHIBITED PANEL

FORCE [kN]

DISPLACEMENT [mm]

COMPRESSION/TENSILE STRESS [MPa]

BUCKLING INHIBITED PANEL

FORCE [kN]

DISPLACEMENT [mm]

COMPRESSION/TENSILE STRESS [MPa]

BUCKLING INHIBITED PANEL

FORCE [kN]

DISPLACEMENT [mm]

COMPRESSION/TENSILE STRESS [MPa]
EXPERIMENTAL CAMPAIGN ON PURE ALUMINIUM SHEAR PANELS
GEOMETRICAL CONFIGURATION OF TESTED PANELS

Panel type F
b/t=50

Panel type B
b/t=100

Panel type G
b/t=50 (=25 in the corners)

Panel type H
b/t=50 (ribs: UPN50)
CYCLIC BEHAVIOUR OF TESTED PANELS

Panel type F

Panel type B

Panel type G

Panel type H
Shear panel “Type B”

Shear panel “Type F”

NUMERICAL SIMULATION OF EXPERIMENTAL TESTS ON ALUMINIUM PANELS
ANALYTICAL INTERPRETATION OF EXPERIMENTAL TESTS

**HARDENING RATIO**

\[ \tau_{\text{max}} / \tau_{02} \]

\[ E_S = \frac{1}{8} \Delta \tau \cdot \Delta \gamma \]

\[ \zeta_{eq} = \frac{1}{4\pi} \frac{E_D}{E_S} \]

\[ G_{sec} = \frac{\Delta \tau}{\Delta \gamma} \]

**AW 1050A-Type H**
ANALYTICAL INTERPRETATION OF EXPERIMENTAL TESTS

SECANT SHEAR STIFFNESS

\[ G_{\text{sec}} = \frac{\Delta \tau}{\Delta \gamma} \]

\[ E_S = \frac{1}{8} \Delta \tau \cdot \Delta \gamma \]

\[ \zeta_{eq} = \frac{1}{4\pi} \frac{E_D}{E_S} \]
ANALYTICAL INTERPRETATION OF EXPERIMENTAL TESTS

EQUIVALENT VISCOUS DAMPING FACTOR

\[ E_S = \frac{1}{8} \Delta \tau \cdot \Delta \gamma \]

\[ \zeta_{eq} = \frac{1}{4\pi} \frac{E_D}{E_S} \]

\[ G_{sec} = \frac{\Delta \tau}{\Delta \gamma} \]

\( \approx 35\% \)

AW 1050A-Type H

AW 1050A-Type G
GEOMETRICAL CONFIGURATION OF TESTED SHEAR PANELS

BTPASP configuration 1
(b/t=100)

SHEAR CYCLIC TESTS BY DIAGONAL STRESS

BTPASP configuration 2
(b/t=50)
GLOBAL EXPERIMENTAL RESULTS

[Images of experimental results and configurations, showing graphs and images of hardware configurations.]
SEISMIC PROTECTION OF EXISTING R.C. BUILDINGS BY METAL SHEAR PANELS

ILVA-IDEML (Intelligent DEMolition) RESEARCH PROJECT
Coordinator: prof. F.M. Mazzolani - University of Naples “Federico II”

Module n°5
Metal shear panels
Other than the structural degradation due to both age and environmental conditions, the experimental test on SMA braces in the transversal direction determined a further reduction of the module mechanical features. Such a situation has been considered in the definition of the numerical model of the sub-structure, which has been used in order to calibrate the structural cyclic experimental behaviour in the longitudinal direction, where seismic retrofitting intervention has been foreseen.

THE BUILDING UNDER INVESTIGATION

Structural consolidation in the transverse direction

Reaction steel structure used in the experimental activity

The experimental test
Starting from the knowledge of the contribution which shear panel should provide in terms of both strength and stiffness, its design can be performed by means of the following simplified theoretical relationships:

\[ V = \frac{1}{2} f_t b L \sin 2\alpha \quad \text{and} \quad K = \frac{E b \cdot t}{4 h_s} \]

Shear panels are realised with two different metallic materials:

- **STEEL**
  - \( f_y = 305 \text{ N/mm}^2 \)
  - \( f_u = 340 \text{ N/mm}^2 \)
  - \( \varepsilon_u > 30\% \)

- **PURE ALUMINIUM**
  - \( f_{02} = 20 \text{ N/mm}^2 \)
  - \( f_u = 80 \text{ N/mm}^2 \)
  - \( \varepsilon_u > 30\% \)

The shear wall (600x400mm) connected to the frame by means of M14 bolts having pitch of 50 mm.

Being \( b/d = 0.25 \) (less than the minimum value (0.8) which guarantees the full activation of tension field), in order to increase such a ratio, two coupled 100 x 4 mm fishplates have been inserted on both sides of the plate.
SEISMIC RETROFITTING METHODOLOGY AND DESIGN OF RC STRUCTURES BY MEANS OF METAL SHEAR PANELS

THE EXTERNAL STEEL FRAME
SEISMIC RETROFITTING METHODOLOGY AND DESIGN OF RC STRUCTURES BY MEANS OF METAL SHEAR PANELS

THE STEEL FRAME – RC STRUCTURE CONNECTIONS
From the cyclic test, it is apparent that the loading phases stiffness are not coincident with the unloading phases one due to the formation of foldings in the upper sub-panel which make the system less rigid and delay the activation of the tension field. Full plastic behaviour of all sub-panels with permanent deformations. Tension field develops in the plates, but the folding is always present. Very pronounced buckling waves which interest all panel fields (folding disappears). Creation of buckling wave in the upper sub-panel.
- quasi-static conditions;
- increase of 10 kN for each cycle up to the last one, where the increment was of 60 kN;
- symmetric load condition up to 200 kN (hence the compression load has been increased up to 300 kN only).

As in the experimental test on the structure retrofitted with steel panels, also in this case the presence of buckling waves determine, even if in less pronounced way, a reduction of the system stiffness. No buckling phenomena were observed. Pronounced foldings in the end sub-panels. Buckling waves in the panel. Foldings increase in the terminal panel fields.