Theoretical and Applied Mechanics - AIMETA 2022 Materials Research Forum LLC
Materials Research Proceedings **26** (2023) 157-162 https://doi.org/10.21741/9781644902431-26

Innovative devices for the protection of welded sections in steel structures

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Keywords: Steel Structures, Moment Resisting Connections, Innovative Devices

Abstract. The paper proposes the use of innovative devices devoted to the brittle collapse protection of welded steel sections, typically represented by the end beam cross-sections in framed structures. Reference is made to I-shaped cross-sections. At first, limiting to the case of plane stress, the relevant elastic domain is defined in the N, T, M space; then a plane frame equipped with the proposed devices and subjected to seismic load condition is studied, ensuring that the generalized stresses at the welded sections be within the relevant elastic domain.

Introduction

As it is well known, in the design of framed steel structures, special attention must be paid to the end beam sections connected to the columns. These cross-sections are usually welded to steel plates bolted to the flange columns. The welding can produce a modification of the material crystal lattice and, consequently, the transition from a ductile behaviour to a brittle one. Therefore, it is cautious to make suitable elastic check for the welded cross-sections adopting appropriate safety factors to avoid undesired brittle collapse.

A recommended strategy consists of limiting the stresses acting on the welded cross-sections by making use of special innovative devices [1-8] for beam-column connection, able to preserve the node integrity without modifying the elastic behaviour. As widely reported in [7-8], the proposed devices possess the property of suitably reducing the generalized stress conditions at the beam extreme maintaining unaltered the elastic bending stiffness. This latter feature makes the device different from the usual adopted ones available for structural designer [9-13] and approved by international codes (see e.g. [14-15]). In the present paper, referring to classical I-shaped profiles, the elastic domain of the relevant cross-sections is firstly defined, described in a suitable analytic form and represented in the N, T, M space. Then the improved optimal design problem is proposed and utilized for a simple plane frame, confirming the full reliability of the procedure and the goodness of the new optimal design formulation.

N, T, M elastic domain of a I-shaped cross-section

Referring to a typical I-shaped cross-section in the plane y, z of the principal axes of inertia (with y the greater inertia axis), the limit elastic axial force, shear force and pure bending moment have the form, respectively,

$$N^E = A\sigma_b; \quad T^E = \frac{\sigma_b I_y a}{2S_y'(G)}; \quad M^E = W_y^E \sigma_b \tag{1}$$

with A cross-section area, σ_b expected limit brittle stress evaluated as a fraction of the relevant material yield stress, I_y cross-section moment of inertia with respect to the y axis, a cross-section web thickness, $S_y'(G)$ statical moment of half cross-section with respect to the y axis, W_y^E cross-section elastic resistance modulus with respect to the y axis. In defining T^E, $\tau_b = \sigma_b/2$ has been assumed, being τ_b the chosen shear limit stress.

According with the Tresca yield criterion, the elastic domain boundary on the first quarter of the N, M plane is defined by the function

$$\frac{N}{A}+\frac{M}{W_y^E}=\sigma_b \tag{2}$$

holding for $0 \leq N \leq N^E$ e $0 \leq M \leq M^E$. On the other quarters the boundary can be defined imposing symmetry with respect to the N and M axes (Fig. 1a).

The elastic domain boundary on the first quarter of the N, T plane is defined by the function:

$$\left(\frac{N}{A}\right)^2+4\left[\frac{TS_y'(G)}{I_y a}\right]^2=\sigma_b^2 \tag{3}$$

holding for $0 \leq N \leq N^E$ and $0 \leq T \leq T^E$. On the other quarters the boundary can be defined imposing symmetry with respect to the N and T axes (Fig. 1b).

The elastic domain boundary on the first quarter of T, M plane is defined by combining the function (flange maximum stress value):

$$\left(\frac{M}{W_y^E}\right)^2+4\left[\frac{TS_y'(E)}{I_y e}\right]^2=\sigma_b^2 \tag{4}$$

holding for $0 \leq T \leq T^E$ and $0 \leq M \leq M^E$, and the discrete set of couples of values of shear and bending moment obtained by the solution to the minimum problem (web maximum stress value):

$$\min_{(z)} M \qquad \text{subjected to}$$
$$\left(\frac{M}{I_y}z\right)^2+4\left[\frac{\overline{T}S_y'(z)}{I_y a}\right]^2 \geq \sigma_b^2 \tag{5}$$
$$0 \leq z \leq \left(\frac{h}{2}-e\right)$$

for a prefixed discrete set of values of $\overline{T}$ $\left(0 \leq \overline{T} \leq T^E\right)$. On the other quarters the boundary can be defined imposing symmetry with respect to the T and M axes (Fig. 1c).

The elastic domain boundary surface on the first octant of N, T, M space is defined by combining the function (flange maximum stress value):

$$\left(\frac{N}{A}+\frac{M}{W_y^E}\right)^2+4\left[\frac{\overline{T}S_y'(E)}{I_y e}\right]^2=\sigma_b^2 \tag{6}$$

which provides a discrete set of functions N, M in correspondence to an analogous discrete set of values of $\overline{T}$ $\left(0 \leq \overline{T} \leq T^E\right)$, and the discrete set of values of axial force, shear force and bending moment obtained by the solution to the minimum problem (web maximum stress value):

$$\min_{(z)} T \qquad \text{subjected to}$$
$$\left(\frac{\overline{N}}{A}+\frac{\overline{M}}{I_y}z\right)^2+4\left[\frac{TS_y'(z)}{I_y a}\right]^2 \geq \sigma_b^2 \tag{7}$$
$$0 \leq z \leq \left(\frac{h}{2}-e\right)$$

for a prefixed discrete set of couples of values of $\overline{N}$ and $\overline{M}$, with $0 \leq \overline{N} \leq N^E$ and $0 \leq \overline{M} \leq M^E$. The domain boundary is completed by symmetry with respect the coordinate planes (Fig. 1d).

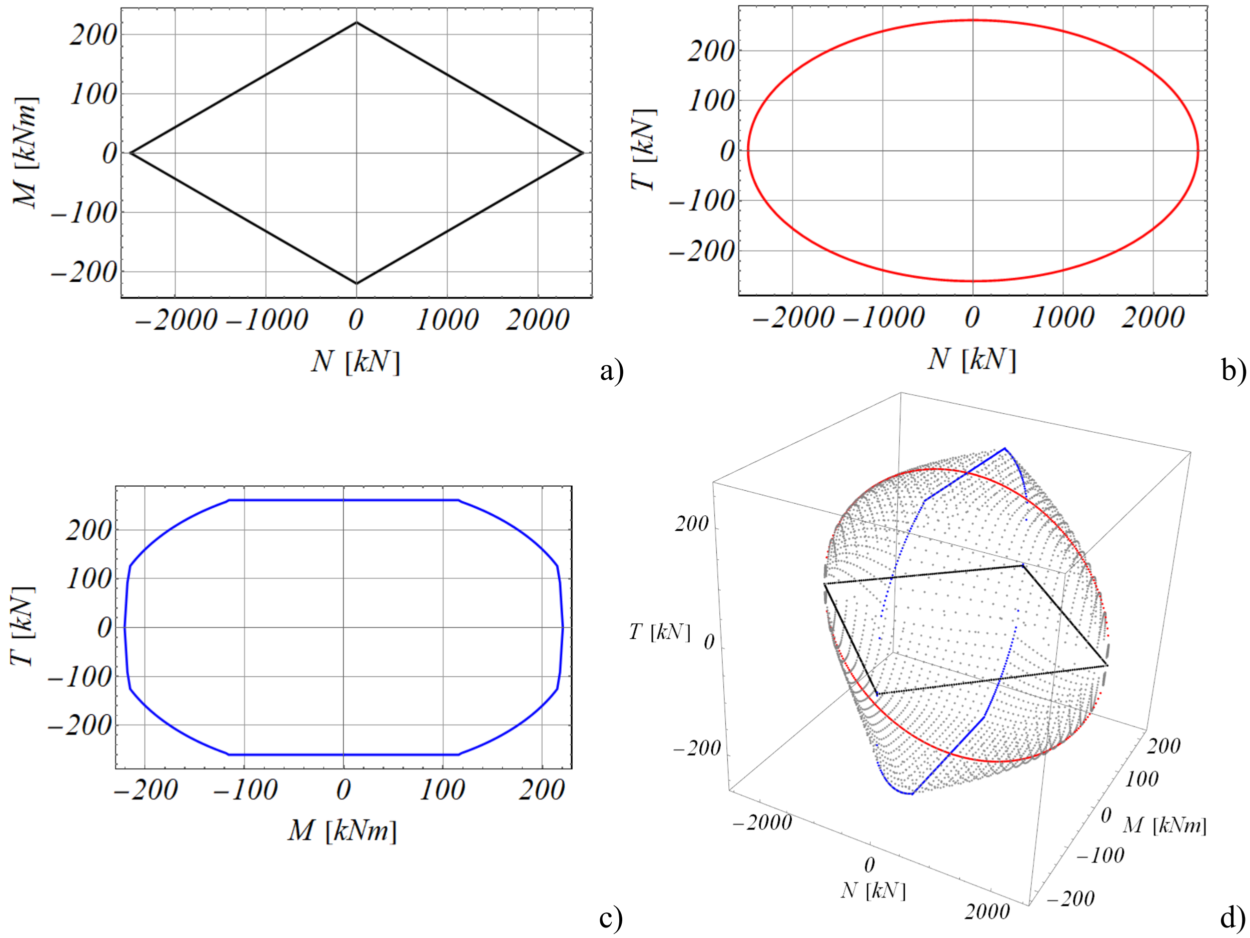

Fig. 1 – HEB240 profile (S235) elastic domain: a) N, M plane; b) N, T plane; c) M, T plane; d) N, M, T space.

Application

Let us consider the plane frame sketched in Fig. 2.

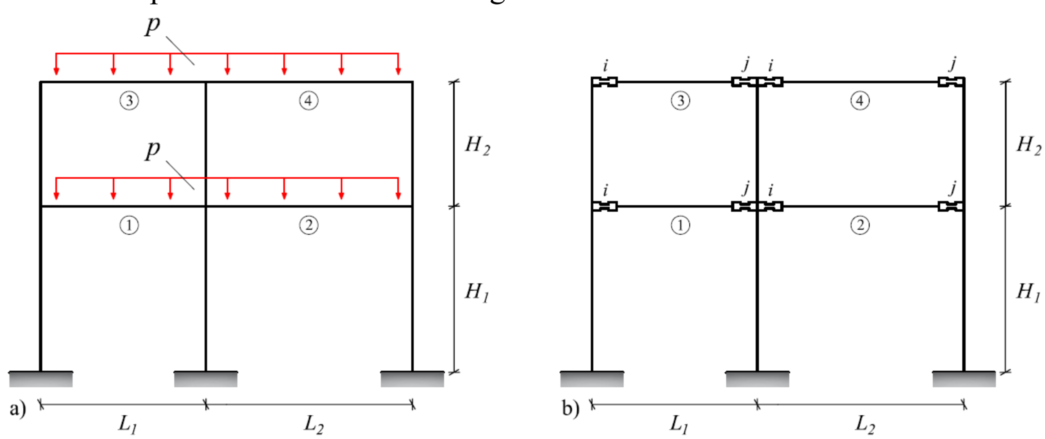

Fig. 2 – Steel plane frame: a) geometry and load condition; b) position of the LRPD.

The material constituting the frame is a steel S275 grade ($E = 210.000$ MPa). The geometrical data are: $L_1 = 4$ m; $L_2 = 5$ m; $H_1 = 4$ m; $H_2 = 3$ m. The distributed load $p = 45$ kN/m is the sum of $G_1 = 18$ kN/m (dead load), $G_2 = 12$ kN/m (permanent loads) and $Q_k = 15$ kN/m (variable load). Beams 1 and 3 are constituted by HEA220 profiles, beams 2 and 4 are constituted by HEA260 profiles. The columns are constituted by HEB340 profiles.

At first, a modal dynamic analysis has been developed for the structure with behaviour factor $q = 4$, verifying the compliance of the ductile response of the element structure with the considered Italian code. Then the same analysis has been developed with behaviour factor $q = 1$. In this last case it has been verified, as expected, that the beam element extremes suffer load conditions above the related elastic domain. In Table I are reported the generalized stress values (N, T, M) in correspondence of all the beam element extremes and the related elastic limit bending moment value M^d for the assigned couple of N and T considering $\sigma_b = 0.9\ \sigma_0$.

***Table I**. Generalized stress response and limit bending moments for the beam end cross-section.*

	Beam internal forces							
	1 *i*	1 *j*	2 *i*	2 *j*	3 *i*	3 *j*	4 *i*	4 *j*
N	67.321	39.657	45.332	60.18	47.037	53.793	64.156	76.68
T	127.624	135.362	164.046	157.34	120.449	135.151	165.671	153.829
M	15,211.25	15,618.51	22,889.04	22,459.2	13,644.86	14,437.84	20,676.48	19,864.96
M^d	**10,672.07**	**10,939.92**	**17,911.30**	**18,306.07**	**11,661.63**	**10,359.52**	**17,691.75**	**18,270.61**

The values $N_{\alpha k}, T_{\alpha k}$ and $M_{\alpha k}^d$ for $\alpha = 1,2,3,4$ and $k = 1,2$ have been utilized for designing the LRPD devices (for the specific geometry details see [1-8]). Imposing, as usual, for the internal portion length $\ell_i = 0.5h$, with h indicating the height of the relevant cross-section, the optimal design problem results (see [7-8]) are listed in the Table II.

***Table II**. Optimal design results.*

	LRPD Optimal dimensions							
	1 *i*	1 *j*	2 *i*	2 *j*	3 *i*	3 *j*	4 *i*	4 *j*
h^*	18.0887	18.0725	21.8816	21.984	18.4181	17.5866	21.8009	22.0183
$t_{f,o}$	2.9113	2.9275	3.1184	3.016	2.5819	3.4134	3.1991	2.9817
$t_{f,i}$	1.1	1.1	1.25	1.25	1.1	1.1	1.25	1.25
ℓ_o	15.8804	15.4395	16.1619	14.8654	11.6727	19.5643	17.2592	14.4467
bi	10.3949	10.595	13.7983	14.5868	12.7731	8.9274	13.1891	14.8587

The considered frames were both studied by performing a non-linear dynamic analysis subjected to an assigned seismic time history compatible with the response spectrum (Fig. 3).

In Fig. 4 the bending moment response function related to the extremes of beam elements 2 and 3 is plotted where the dashed lines represent the limit bending moment imposed to avoid any undesired brittle behaviour. As it is possible to observe, the original frame exhibits a generalized stress response that in many instants is higher than the prescribed assigned limit value, resulting above the elastic domain previously defined. Whereas, as it was expected, the frame equipped with the suitably designed devices, exhibits a generalized stress response substantially brittle safe. The rare instants in which the prescribed limit is exceeded are due to the evident difference between the spectral analysis and compatible seismic time history and they can be accepted from a practical point of view or avoided by imposing more stringent brittle safety factor.

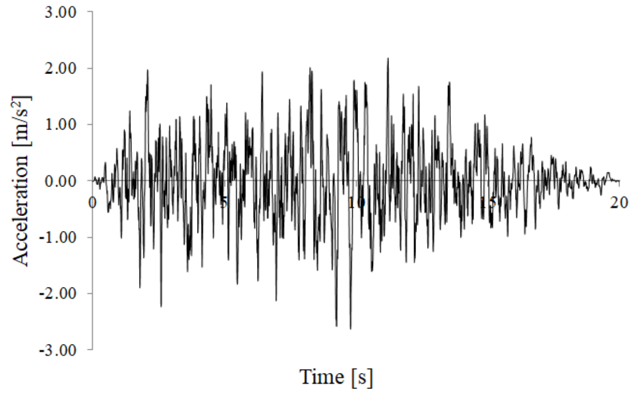

Fig. 3 – Accelerogram applied to the studied frames.

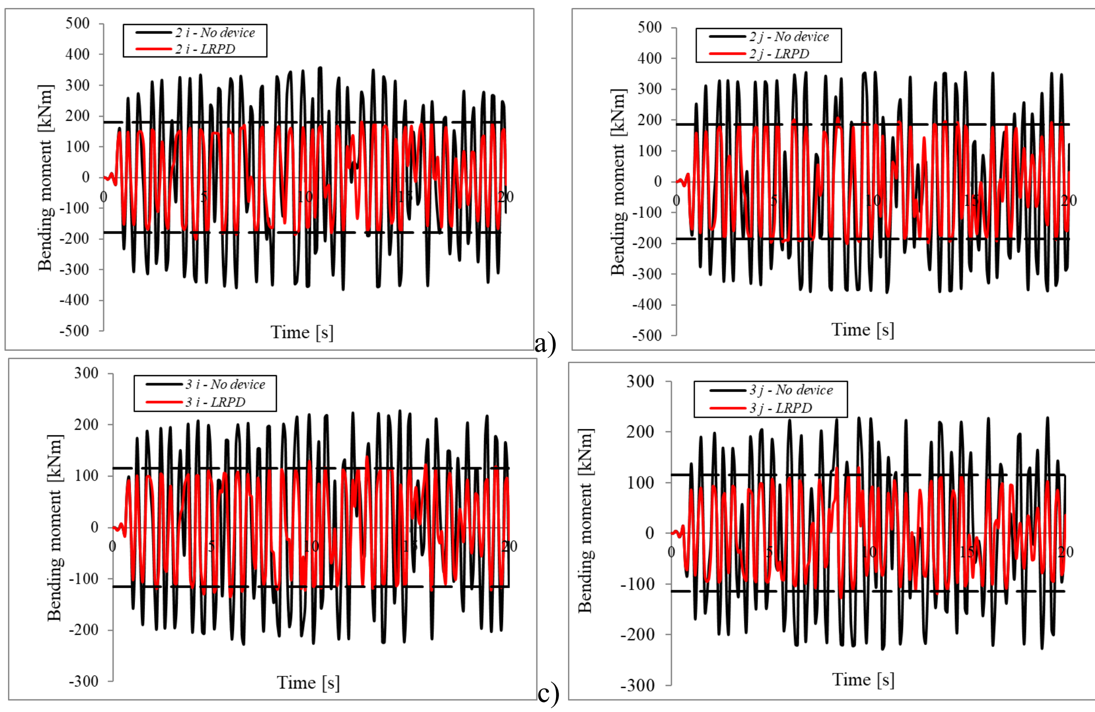

Fig. 4 – Bending moment response evaluated at the extremes of beams 2 and 3.

Conclusions

In the present paper, a special strategy devoted of limit the generalized stresses acting on the welded cross-sections of a frame structure by making use of some innovative devices for beam-column connections, able to preserve the node integrity without modifying the elastic behaviour, is proposed.

The computational procedure consists of evaluating the limit elastic bending moment on the relevant cross-sections complying the reference N, T, M domain and designing the LRPD devices for the assigned limit stress values able to prevent brittle behaviour as a percentage of elastic limit stress suitably selected depending on the welding methodology. The performed numerical applications, related to a simple plane frame, confirm the effectiveness of the proposed strategy.

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