Plastic design of aluminium-concrete composite sections: a simplified method

Autor(en): **Mandara, Alberto / Mazzolani, Federico M.**

Objekttyp: **Article**

Zeitschrift: **IABSE reports = Rapports AIPC = IVBH Berichte**

Band (Jahr): **999 (1997)**

PDF erstellt am: **03.05.2024**

Persistenter Link: https://doi.org/10.5169/seals-993

Nutzungsbedingungen

Die ETH-Bibliothek ist Anbieterin der digitalisierten Zeitschriften. Sie besitzt keine Urheberrechte an den Inhalten der Zeitschriften. Die Rechte liegen in der Regel bei den Herausgebern.
Die auf der Plattform e-periodica veröffentlichten Dokumente stehen für nicht-kommerzielle Zwecke in Lehre und Forschung sowie für die private Nutzung frei zur Verfügung. Einzelne Dateien oder Ausdrucke aus diesem Angebot können zusammen mit diesen Nutzungsbedingungen und den korrekten Herkunftsbezeichnungen weitergegeben werden.
Das Veröffentlichen von Bildern in Print- und Online-Publikationen ist nur mit vorheriger Genehmigung der Rechteinhaber erlaubt. Die systematische Speicherung von Teilen des elektronischen Angebots auf anderen Servern bedarf ebenfalls des schriftlichen Einverständnisses der Rechteinhaber.

Haftungsausschluss

Alle Angaben erfolgen ohne Gewähr für Vollständigkeit oder Richtigkeit. Es wird keine Haftung übernommen für Schäden durch die Verwendung von Informationen aus diesem Online-Angebot oder durch das Fehlen von Informationen. Dies gilt auch für Inhalte Dritter, die über dieses Angebot zugänglich sind.

Ein Dienst der *ETH-Bibliothek*
ETH Zürich, Rämistrasse 101, 8092 Zürich, Schweiz, www.library.ethz.ch

http://www.e-periodica.ch

Plastic Design of Aluminium-Concrete Composite Sections: a Simplified Method

Alberto MANDARA
Eng.
University of Naples
Naples, Italy

Alberto Mandara, born 1963, is Research Assistant at the Second University of Naples. Graduated in 1987, he got the PhD degree in 1993. Author of more than 40 papers on steel and aluminium structures. Presently member of UNI-CIS/SC3, "Steel and Composite Structures" and UNI-CIS/SC10 "Structural Restoration" and CEN-TC 250/SC9 " Aluminium Alloy Structures".

Federico M. MAZZOLANI
Prof. Dr.
University of Naples
Naples, Italy

Federico M. Mazzolani, born 1938, is Full Professor of Structural Engineering at the University "Federico II" of Naples. Author of more than 350 papers and 12 books in the field of metal structures, seismic design and rehabilitation. Member of many national and international organisations. Presently Chairman of UNI-CIS/SC3 "Steel and Composite Structures", CNR Fire Protection", ECCS-TC13 "seismic "Design" and CEN-TC 250/SC9 "Aluminium Alloy Structures".

Summary

A practical procedure for the design of aluminium-concrete composite sections in bending is presented in this paper. The actual inelastic behaviour of materials is considered for the evaluation of section ultimate load bearing capacity. A numerical analysis has been performed for evaluating the response of aluminium alloy section, whereas the existing CEB regulations have been applied for the prediction of concrete inelastic response.

1. Foreword

Despite several applications in the field of bridge structures in France and in U.S.A., aluminium-concrete composite structures are not yet covered by specific regulations, neither in Europe, nor overseas. At the present stage of knowledge, even though the bending behaviour of such structures has been investigated both theoretically and experimentally [1 to 4], no practical method for the evaluation of strength is available in existing codification and designers must face cumbersome numerical calculations to predict both service and ultimate structural response. To some extent, this lack can represent a consequence of the fully non linear behaviour and relatively limited ductility of both concrete and aluminium alloys, which involve some difficulties in the evaluation of ultimate load bearing capacity of the section. In particular, contrary to steel-concrete composite structures, fully plastic idealizations, relying on the assumption of infinitely ductile material, can not be used, because premature collapse of the section may occur due to excess of strain in aluminium alloy. Since in some alloys this failure can be attained for relatively low values of strain, an accurate evaluation of collapse condition must be performed, by taking into account the actual inelastic behaviour and deformation limits of materials. As a consequence, some commonly adopted simplifications, including for example the well known "stress block" approach for the evaluation of concrete ultimate strength, can not be assumed.
On the basis of the existing knowledge in the field of aluminium alloys [5], a method for the prediction of the ultimate strength of composite sections in bending is presented in this paper. The procedure has been fitted on the outcoming of a well tried numerical simulation, which has shown to be in good agreement with experimental results [4]. Since the method is consistent with both CEB regulations on reinforced concrete structures and CEN Eurocode 9 on aluminium alloy structures, it could represent a first attempt to implement this topic into European regulations.

2. Limit state definition

A typical composite aluminium-concrete section is depicted in fig. 1a, where the relevant geometrical parameters are also shown. The slab width B can be the actual one or the effective one B_{eff}, depending on beam geometrical ratios and restraint conditions (see for example EC4 for the evaluation of B_{eff}). G' and G are the geometrical centroids of the aluminium beam and of the whole section, respectively.

For the evaluation of ultimate strength of the section, a preliminary definition of material laws is necessary. The material models adopted are shown in fig. 2. The well known CEB σ-ε law with a parabolic branch up to a strain of .2% and an ultimate strain of .35% is adopted for concrete in compression (fig. 2a). The tensile strength of concrete is neglected. This model has been chosen because it is widely referred to into both literature and codification on r.c. structure; in addition, it makes the evaluation of the plastic behaviour of the concrete slab quite simple, the response of rectangular r.c. section made of this material being extensively tabled in literature.

The classical elastic-plastic σ-ε law is assumed for reinforcement steel bars (fig. 2b). No strain limitation is assumed for steel, neither in tension, nor in compression. Obviously, the maximum deformation in compressed bars must comply with the maximum allowed strain for concrete.

The three-parameter non linear Ramberg-Osgood model $\varepsilon = \sigma / E + 0.002(\sigma / f_{0.2})^n$ has been assumed to interpret the behaviour of aluminium alloys (fig. 2c). In this case, owing to a strain-hardening effect, a perfectly plastic behaviour does not exist, the material response being continuously increasing for a wide range of strain. Since for some alloys the actual deformation capability may result in not very high values of strain, a conventional deformation limit must be set to define the ultimate limit state of material. According to the usually followed approach for the evaluation of ultimate strength of aluminium sections [5], if $\varepsilon_e = f_{0.2} / E$ is the deformation calculated elastically at the conventional yield point $f_{0.2}$, a maximum strain limit $\varepsilon_5 = 5\varepsilon_e$ or $\varepsilon_{10} = 10\varepsilon_e$ can be assumed, depending on the alloy ductility features. The strain limits ε_e and $\varepsilon_e + 0.002$ are also considered, as elastic limit state and conventional yielding limit state, respectively.

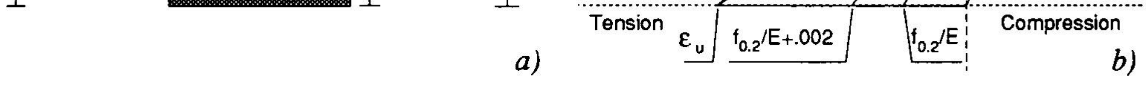


Fig. 1 Geometrical magnitudes (a) and failure conditions (b) of the composite section.

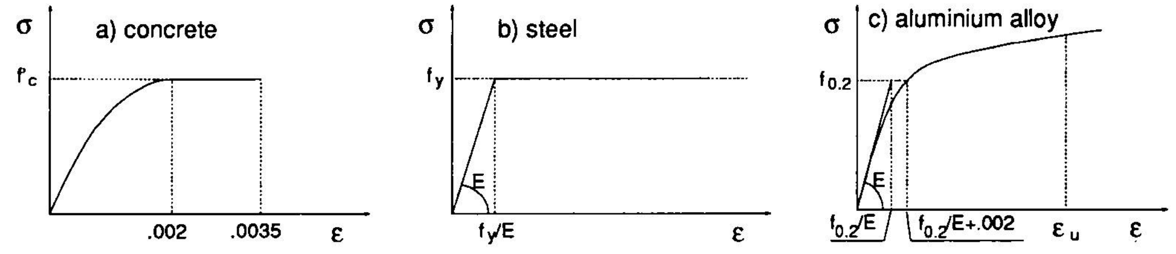


Fig. 2 Material laws

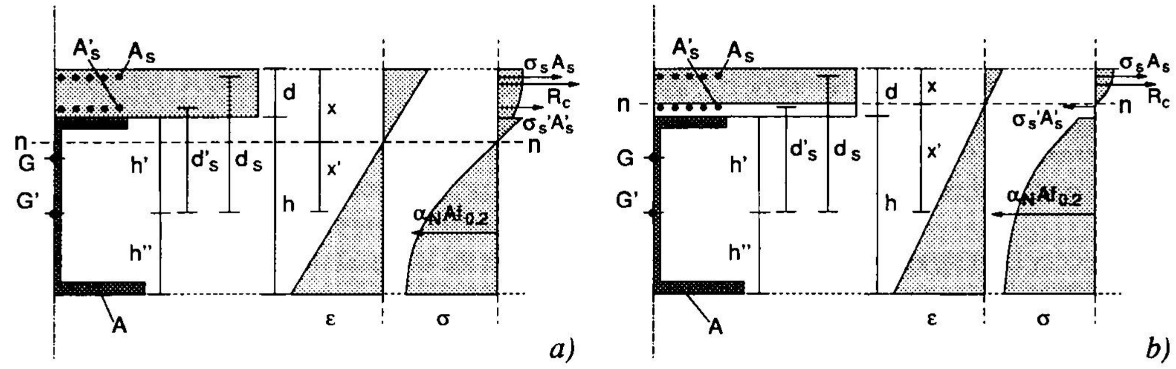


Fig. 3 Distribution of internal actions in the cross-section [a) x > d, b) x < d]

3. Evaluation of ultimate bending strength

Conventionally, it is assumed that the collapse of the section is attained when one of the components reaches the limit value of strain. This may be the aluminium beam and/or the concrete slab, depending on the geometrical and mechanical features of the section. Since generally neither concrete nor aluminium alloy exhibit very large plastic deformations at failure, scarce reliance can be put on the post-elastic strength of material in the evaluation of the plastic response of the section. For this reason, the common approach used for steel-concrete composite sections, based on indefinitely plastic behaviour of material, can not be followed. This involves that many calculation simplifications allowed by the large ductility of steel, such as for example the well known "stress block" idealisation for concrete compressive resultant force, can not be used. Also, the concept of "balanced failure", well known in the literature on r.c. structures, looses most of its meaning.

As being stated, the possible failure conditions of the section, represented in fig. 1b, are exclusively related to the attainment of the above strain limits. Since the assumption of plane cross section up to collapse is made, such conditions are represented by lines, each of which corresponds to a defined location of neutral axis *n-n* at failure (see parameters x or x' in fig. 3). This will fall within the zones marked in fig. 1b: zones A and B correspond to the failure of the aluminium alloy, represented by an ultimate strain ε_u equal to ε_5 or ε_{10}, whereas zones C, D and E represent conditions of concrete collapse by crushing. As for all bending problems, the value of x (or x') is determinated by an equilibrium condition along the longitudinal axis of the beam. According to the notation shown in fig. 3, this condition may be written in the form:

$$A_s\, \sigma_s \pm A'_s\, \sigma'_s + f_c\, Bx\, \Psi - \alpha_N\, f_{0.2}\, A = 0 \qquad (1)$$

The terms in Eq. (1) are shown in fig. 3, in the case when $x > d$ (fig. 3a) and $x < d$ (fig. 3b). They are to be intended in absolute value. The notation "±" means that σ'_s must be taken positive when $d'_s > x'$ and negative when $d'_s < x'$. The magnitude $f_c\, Bx\, \Psi = R_c$ is the resultant of compression force of concrete, Ψ being a nondimensional parameter which takes into account the actual non linear stress distribution in compressed concrete. The values of Ψ are a function of the neutral axis position x/d and can be found in the current literature on r.c. structures for the assumed σ-ε law in the case of rectangular section. The term α_N represents the resultant of normal stresses acting on the aluminium section (see fig. 3); it may be expressed as:

$$\alpha_N = \frac{1}{Af_{0.2}} \int_A \sigma dA \qquad (2)$$

A being the cross sectional area of aluminium beam.

For a given limit state, i.e. for a given strain limit assumed for the alloy at the more stretched fibre of the section, α_N is a function of the σ-ε law of material, of the type of cross section, represented through the geometrical shape factor α_0, as well as of the ratio x'/h.
Similarly, based on the notation of fig. 3, the rotation equilibrium is expressed by the equation:

$$M_u = A_s\sigma_s d_s \pm A'_s\sigma'_s d'_s + f'_c Bx\Psi(d + h' - \lambda x) + \alpha_M f_{0.2} W \qquad (3)$$

The terms in Eq. (3), expressed in absolute value, represent the moment of each internal action evaluated respect to the geometrical centroid of the aluminium section (see Fig. 3). For the sign of σ'_s the same consideration made for N_u holds. λ is a nondimensional parameter which considers the resultant of concrete compressive force in its actual position. As for Ψ, its values can be found in the literature on r.c. structures for the case of rectangular section as a function of x/d. The term α_M is given by the following relationship:

$$\alpha_M = \frac{1}{Wf_{02}}\int_A \sigma z dA \qquad (4)$$

W being the resistance modulus of the aluminium section and z the distance from its centroid.
α_M represents the nondimensional resisting moment of the aluminium section around the geometrical centroid of the double-T profile. As for α_N, α_M is a function of the strain limit assumed for the alloy, of the σ-ε law of material, of the geometrical shape factor α_0 of the section, as well as of x'/h.
For a given deformation limit at the most stretched point of the section, the values of α_N and α_M have been calculated by means of numerical analysis as a function of x'/h. In practice, a value of x'/h is set and then the section overall response in terms of axial action $\int_A \sigma dA$ and resisting moment around the centroid $\int_A \sigma z dA$ is evaluated for each relevant limit state. The values of α_N and α_M are given in Figs 4 and 5 as a function of x'/h for a I-section with geometrical shape factor $\alpha_0 = 1.1$ and for values of the hardening parameter of the Ramberg-Osgood law equal to 8, 16 and 32. Values of limit strain equal to ε_e, $\varepsilon_e + 0.002$, ε_5 and ε_{10} have been considered in order to make possible the evaluation of section response at all relevant limit states.

4. Application of the method

The application of the method is based on the evaluation of the neutral axis depth x (or x') from Eq. (1) and on the subsequent evaluation of M_u from Eq. (3). Since a direct evaluation of x' is not possible from Eq. (1), σ_s, σ'_s, Ψ and α_N being function of x', a trial-and-error procedure must be applied: a tentative value of x is assigned, then, by considering that $x + x' = H - h''$, the obtained value of x is compared with the assumed one and corrected up to obtain the same value given when starting the calculation. All the magnitudes σ_s, σ'_s, Ψ and α_N are to be calculated according to the assumed value of x. If intermediate positions of x between the defined limit states are found, α_N may be evaluated by means of linear interpolation between the curves of fig. 4. This generally occurs when the ultimate limit state of the composite section is attained due to excess of compressive strain into concrete. As a rule, in such cases the deformation at the tensioned edge of the alloy is not equal to any of the defined limit states and, consequently, linear interpolation must be made.
In order to make the calculation easier, a preliminary evaluation of the B values, corresponding to the relevant location of x'/h at the ultimate limit state, can be made. The comparison with the actual value of B will allow to know the range in which x'/h will fall at collapse for the section under consideration.

When the neutral axis position x is known, the calculation of M_u from Eq. (3) is straightforward, being all terms σ_s, σ'_s, Ψ, λ and α_M calculated as a function of x. The evaluation of α_M may be done from fig. 5, directly or by means of linear interpolation, depending on the values of x.
The calculation procedure can be generalised to evaluate the section bearing capacity even for serviceability limit states. Since all limit states are defined by a given material strain limit ε_{al} or ε_c, the corresponding values of section curvature, given by $\chi = (\varepsilon_{al} + \varepsilon_c)/H$, may be calculated. In this way, a piecewise moment-curvature relationship may be plotted for the section.
A comparison between the procedure proposed and the results of a numerical analysis is shown in Fig. 6a, where the M - χ curve is depicted for a composite section made of a double-T extruded aluminium alloy profile with H=200 mm and a 60mm thick concrete slab, reinforced with two rows of Ø10mm bars (Fig. 6b). This example could reasonably represent the case of a floor structure, having main girders spaced 1000mm. The geometrical and mechanical features are shown in Fig. 6b. All material deformation limits referred to in fig. 1b have been considered. The corresponding values of bending moment are marked on the M - χ curve. The values of α_N and α_M have been evaluated approximately assuming $n=16$. The comparison shows a quite satisfying agreement, with results coming from the simplified procedure proposed herein slightly on the safe side with respect to those provided by the more refined simulation approach.

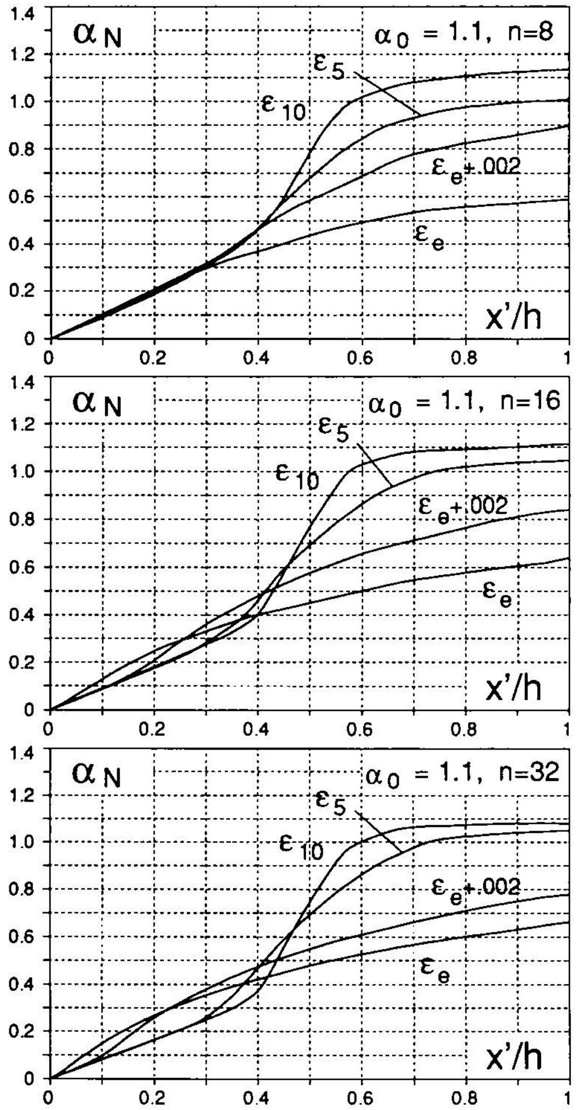


Fig. 4 Values of α_N

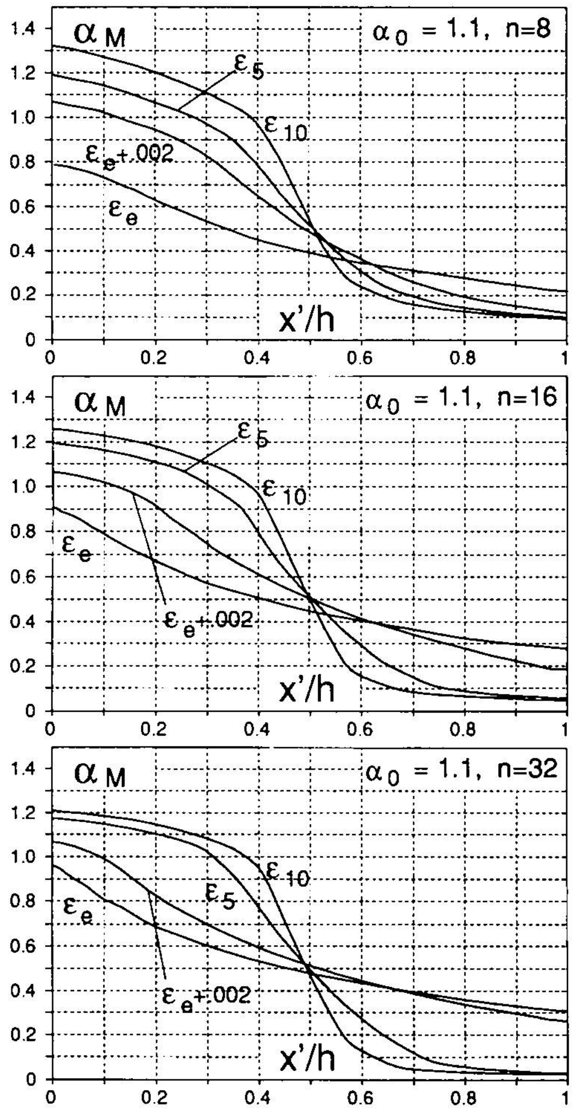
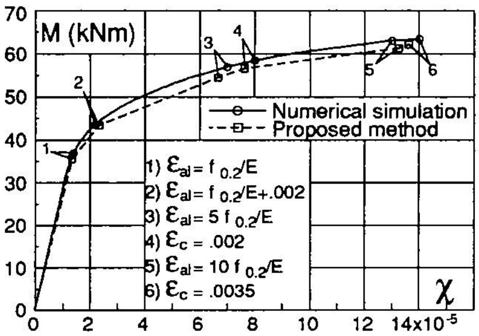
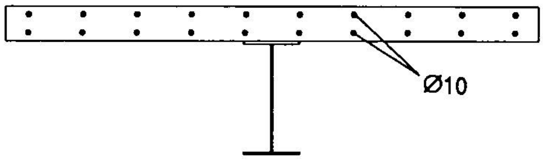


Fig. 5 Values of α_M



a)



H = 260mm	A = 1494mm^2	f'_c = 25N/mm^2
B = 1000mm	A_s = 785mm^2	f_y = 380N/mm^2
d = 60mm	A'_s = 785mm^2	$f_{0.1}$ = 215.6N/mm^2
h' = 100mm	d_s = 140mm	$f_{0.2}$ = 223.6N/mm^2
h" = 100mm	d'_s = 120mm	$n_{R.O.}$ = 19.1

b)

Fig. 6 Comparison of the procedure proposed with numerical simulation.

5. Conclusive remarks

The study presented in this paper represents the logical development of the extensive research work carried out by the Authors on this topic and referred to in [1 to 5]. In those papers, the problem has been faced both from experimental and numerical point of view. Some tests were performed in order to calibrate suitable numerical simulation procedures, which have been successfully used to investigate many aspects of these structures, such as the elastic and post-elastic behaviour, the ductility features, the influence of slab reinforcements, and so on.
The reliability of these simulation methods is the basis of the procedure presented herein. This may be considered as a first approach to the direct estimation of load bearing capacity of aluminium-concrete sections in bending. The difficulty in evaluating the inelastic material response has been overcome through a preliminary numerical study of the aluminium section, which has led to a graphical representation of the section behaviour as a function of the relevant parameters. The existing literature on r.c. structures, including many design manuals, can be profitably used for the prediction of concrete inelastic behaviour. The method proposed is conservative, simple and relatively easy to apply, even though the iterative procedure for the evaluation of the neutral axis position can result in some difficulty when the designer is working without computer aids. Nevertheless, in spite of these achievements, several aspects related to the bending behaviour of this structural typology, namely the effect of the different values of the thermal expansion coefficient, as well as the influence of concrete shrinkage, still remain to be clarified. In addition, the study of the section in simple bending does not exhaust the investigation on the structure regarded as a whole, because the collapse of a composite structure can be also a consequence of other phenomena, such as, for example, the shear brittle failure of the concrete slab, the failure of connectors, the buckling of compressed parts, etc. It is advisable, therefore, that new research can be devoted to this subject in future.

References

[1] Bruzzese, E., Cappelli, M., Mazzolani F.M.: "Experimental Investigation on Aluminium-Concrete Beams", Costruzioni Metalliche, n. 5, 1989.
[2] Bruzzese, E., De Martino A., Mandara A., Mazzolani F.M.: "Esame dei parametri comportamentali del sistema misto alluminio-calcestruzzo", Proc. of the XII C.T.A. Congress, Anacapri, 1989 (in Italian).
[3] Bruzzese, E., De Martino A., Mandara A., Mazzolani F.M.: "Aluminium-Concrete Systems: Behavioural Parameters", Proc. of the International Conference on Steel & Aluminium Structures - ICSAS 91", Singapore, 1991.
[4] Bruzzese, E., Mandara A., Mazzolani F.M.: "Sulla duttilità del sistema misto alluminio-calcestruzzo", Proc. of the XIII C.T.A. Congress, Abano Terme, 1991 (in Italian).
[5] Mazzolani, F.M.: "Aluminium Alloy Structures", FN & SPON, London 1995.