Properties and in-service performance

Practical application of Eurocode 3 to multi-storey buildings with steel 'sway frame' structures

Report
EUR 17879 EN
Practical application of Eurocode 3 to multi-storey buildings with steel ‘sway frame’ structures

F. Fattorini, M. Bandini, F. Cattaneo, S. Pustorino

Centro Sviluppo Materiali
Via di Castel Romano, 100-102
I-00129 Rome

Contract No 7210-SA/419
1 July 1991 to 30 June 1995

Final report
FOREWORD

(1) The research activities described in this report were carried out with the financial assistance of the European Coal and Steel Community (ECSC).

(2) Particular thanks for fruitful collaboration are addressed to:

- Mr. A. Bureau - Mr. B. Chabrolin
- Mr. P. Chantrain
- Prof. R. Zandonini

CTICM (F)
ProfilARBED-Recherches (L)
University of Trento (I)

(3) Grateful thanks are also expressed to:

- Prof. E. Bozzo
- Prof. A. De Luca
- Prof. G. Macchi
- Prof. P. E. Pinto
- Mr. S. Pustorino
- Prof. H. Rubin
- Prof. G. Sedlacek - Mr. D. Grotmann
- Mr. C. Taylor

SIDERCAD (I)
University of Basilicata (I)
University of Pavia (I)
University of Rome (I)
CREA (I)
TU (A)
RWTH (D)
SCI (GB)
Abstract

The present work deals with the analysis and verification, according to the Eurocodes, of a particular class of steel structures known as "sway frames" in which the nodal horizontal displacements produce non negligible additional internal moments (P-Δ effects) that have to be summed up to those deriving from a first order analysis.

In the design of these structures, the actions are determined in accordance with the EC1; EC8 recommendations are used when the seismic effects have to be taken into account.

The final result of this project is the draft of a practical design "handbook" for structural engineers dealing with the design of steel "sway frame" structures according to EC3. The contents of this document are the design calculation rules for this category of structures. Besides, some indications are provided as to the use of popular general purpose computer codes available in the European market for a reliable and efficient design of "sway frame" systems. Finally, a number of calculation examples of 2D-frames have been included.

From a practical point of view this research has evidenced that common types of steel structures as industrial buildings, multi-storey parkings, warehouse buildings and small civil buildings subject to usual vertical loads may fail to fulfil the "non sway" criterion and thus have to be defined as "sway frames".

The results have shown that for asymmetrical plane frames the rules of the EC3 for the simplified "sway-non sway" classification and for the indirect second order elastic analysis methods have to be reconsidered in order to make them more clear and to improve their level of accuracy. Further studies are needed to develop the background to simple yet reliable design criteria.

In particular, the activities performed during this work were divided in six phases:

Phase 1: Study of the EC3, EC1, and EC8, with particular reference to frame design;

Phase 2: Study and comparison of some of the major national standards regarding steel structures, with special attention to the clauses regarding "sway frame" analysis;

Phase 3: Identification of second and first order general purpose computer codes for "sway frame" analysis;
Phase 4: Selection of a number of buildings of interest, in an European context, for the calculation examples. The calculation of these structural models according to EC3 methodologies and the subsequent member checks. Regarding these local checks, a link with C.T.I.C.M.'s project SA/312 has been obtained performing a number of local verifications with the code "EC3 Tools" developed during the research. Additional calculations have been performed according to some of the National standards studied during phase 2;

Phase 5: A conservative assessment of the results of the calculation examples performed in phase 4. The features of the methods adopted in the calculation phase have been extensively illustrated.

Phase 6: The draft of a practical design handbook for civil engineers regarding the "sway frame" elastic analysis and checks according to EC3. It contains detailed design procedures for these structures and indications on the main characteristics of popular general purpose computer codes. The general lines of this manual have been decided in collaboration with the researchers of ProfilARBED Recherches working on project SA/513. The aim is to obtain a link between the two works, taking into account that the two projects deal with structures that belong to the "family" of the steel frames.
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - INTRODUCTION</td>
<td>9</td>
</tr>
<tr>
<td>2 - EUROPEAN STANDARDS AND RECOMMENDATIONS</td>
<td>11</td>
</tr>
<tr>
<td>2.1 - EUROCODE 3</td>
<td>11</td>
</tr>
<tr>
<td>2.1.1 - Limit states and design situations (EC3 section 2.2)</td>
<td>11</td>
</tr>
<tr>
<td>2.1.2 - The Serviceability Limit States (SLS)</td>
<td>12</td>
</tr>
<tr>
<td>2.1.3 - The Ultimate Limit States (ULS)</td>
<td>12</td>
</tr>
<tr>
<td>2.1.4 - The Connections</td>
<td>15</td>
</tr>
<tr>
<td>2.2 - EUROCODE 1</td>
<td>17</td>
</tr>
<tr>
<td>2.3 - EUROCODE 8</td>
<td>17</td>
</tr>
<tr>
<td>2.3.1 - Part 1.1</td>
<td>17</td>
</tr>
<tr>
<td>2.3.2 - Part 1.2</td>
<td>18</td>
</tr>
<tr>
<td>2.3.3 - Part 1.3</td>
<td>18</td>
</tr>
<tr>
<td>2.4 - ECCS PUBLICATION No.33</td>
<td>19</td>
</tr>
<tr>
<td>2.4.1 - Definitions</td>
<td>20</td>
</tr>
<tr>
<td>2.4.2 The analysis methods</td>
<td>20</td>
</tr>
<tr>
<td>3 - THE NATIONAL STANDARDS</td>
<td>22</td>
</tr>
<tr>
<td>3.1 - GENERAL</td>
<td>22</td>
</tr>
<tr>
<td>3.2 - UNITED KINGDOM</td>
<td>22</td>
</tr>
<tr>
<td>3.2.1 - General</td>
<td>22</td>
</tr>
<tr>
<td>3.2.2 - &quot;Sway-Non Sway&quot; criterion</td>
<td>23</td>
</tr>
<tr>
<td>3.2.3 - Calculation methods for &quot;sway frames&quot;</td>
<td>24</td>
</tr>
<tr>
<td>3.3 - FRANCE</td>
<td>25</td>
</tr>
<tr>
<td>3.3.1 General</td>
<td>25</td>
</tr>
<tr>
<td>3.3.2 The &quot;Additif 80&quot;</td>
<td>25</td>
</tr>
<tr>
<td>3.4 - ITALY</td>
<td>26</td>
</tr>
<tr>
<td>3.4.1 General</td>
<td>26</td>
</tr>
<tr>
<td>3.4.2 &quot;Sway-Non Sway&quot; criterion</td>
<td>26</td>
</tr>
<tr>
<td>3.5 - GERMANY</td>
<td>27</td>
</tr>
<tr>
<td>3.5.1 - General</td>
<td>27</td>
</tr>
<tr>
<td>3.5.2 - &quot;Sway-Non Sway&quot; criteria</td>
<td>27</td>
</tr>
<tr>
<td>3.5.3 - Elastic analysis of &quot;sway frames&quot;</td>
<td>28</td>
</tr>
<tr>
<td>3.6 - SWITZERLAND</td>
<td>29</td>
</tr>
<tr>
<td>3.7 - USA</td>
<td>29</td>
</tr>
<tr>
<td>3.8 - JAPAN</td>
<td>29</td>
</tr>
</tbody>
</table>
1 - INTRODUCTION

The design analysis of frames is extensively covered by Eurocode 3, which provides design methods at different levels of complexity and accepts recent trends with reference to frame classification, imperfection and types of analysis.

ProfilARBED recherches' research project SA/513 "Simplified version of the Eurocode 3 for usual buildings" deals with the design of common buildings with bracing structures (steel members or r.c. elements) whose behaviour, with respect to the second order effects, is "non sway".

A gap remained for the structures that did not comply the "non sway" criterion; the development of the present work, dealing with the analysis of "sway frames", is based on this need.

Being the calculation methods for this type of structures not so familiar to most of the designers of steel structures, it was thought that for a full comprehension of the design approaches suggested by the EC3 it would have been of primary importance to examine some of the available documentation and publications on the analysis of "sway frames".

Following this line, a number of European and non European publications have been examined: these refer to the global stability checks of frames, the simplified methodologies for the "sway-non sway" criterion, the calculation of the elastic critical load factor, the correct interpretation of the clauses contained in the Eurocode for "sway frame" analysis, etc.

The fact that most of these papers and documents have been published in the last years is a clear index that the problem of the calculation approaches which account for geometric effects is very actual and closely related to the progresses made by the computer codes in the last decade.

A review of these publications has been performed. Among these, the ECCS N.33 [18] should be mentioned; as it is illustrated in section 2.4 in the document are present a number of methods for the calculation of "sway frames".

In [47] Horne describes a method for estimating the "elastic critical load factor" of an unbraced multi-storey frame. The document is the basis for the simplified "sway-non sway" criterion given in Eurocode 3 and apparently has a validity for every type of frame; nevertheless, it is worth mentioning that the calculations to which the publication refers have been performed only on symmetrical frames symmetrically loaded by the vertical loads. For this reason, the method could show to be inadequate for asymmetrical structures.

De Luca, in [27], performs a comparison of the calculation methods for "sway frames" of the EC3 and of the Italian CNR 10011. The author suggests a number of modifications on the "sway mode buckling lengths" method of the Eurocode and evidences that in relation to the CNR the code has a much more complete approach.
In [26] Bozzo evidences that the approach proposed by the Italian CNR in introducing limits for the horizontal deflections referring to the whole structure is incorrect and that the criteria should refer to each single storey.

De Luca, in [49], gives tabular methods for the determination of the "elastic critical vertical load" of the frame.

Many other publications have been examined with the intent to clarify other aspects of the calculation of "sway frames".

To complete this initial "study phase", other National standards on steel structures have been examined and compared to the EC3 with particular attention to the rules regarding the geometric non linearities in a structural analysis.
2 - EUROPEAN STANDARDS AND RECOMMENDATIONS

In this chapter will be briefly illustrated some of the main clauses of the EC3, EC1 and EC8 dealing with the analysis of rigid frames with a "sway" behaviour. Furthermore, a concise description of the ECCS publication No.33 has been included.

In this work the following editions of the Eurocodes have been considered:

- EC1: Draft - "Basis of design and actions on structures" - December 1992;

2.1 - EUROCODE 3

2.1.1 - Limit states and design situations (EC3 section 2.2)

These are states beyond which the structure no longer satisfies the design performance requirements. The Eurocode prescribes two different classes of limit states:

- Ultimate Limit States: are those associated with the collapse or with other forms of structural failure which may endanger the safety of people. In particular are considered:

  * loss of equilibrium of the structure or any part of it;
  * failure by excessive deformation;
  * failure by rupture;
  * failure by loss of stability of the structure or any part of it.

- Serviceability Limit States: are those corresponding to states beyond which specific service criteria are no longer met. In particular are considered:

  * deformations or deflections which adversely affect the appearance or use of the structure or cause damage to finishes or non-structural elements;
  * vibrations which cause discomfort to the people, damage the building or its contents or which limit its functional effectiveness.
2.1.2 - The Serviceability Limit States (SLS)

In particular, the EC3 requires SLS verifications (see chapter 4): for the horizontal and vertical deflections of the frame in which due allowance should be made for the second order effects. The limits and checks given in this section of the Eurocode are illustrated in detail in the "design handbook" for engineers.

In addition, for structures subject to loads that can induce impact or vibration, design criteria regarding the minimum natural frequency of the building have been introduced.

2.1.3 - The Ultimate Limit States (ULS)

All the major rules regarding the definition, classification and analysis of "sway frames" have been included in chapter 5 of the EC3.

In this paragraph, a brief description of some of the design phases is given. In any case, the full sequence is given in the design handbook.

1) Calculation of the internal forces and moments (section 5.2)

The methodologies of analysis given in this part of the standard depend entirely on the behaviour of the structure with reference to the second order effects; if the frame is classified as "non sway", ordinary first order calculation methods are sufficient to calculate the internal forces. On the other hand, if the structure is "sway", it is necessary to take into account either directly or indirectly the second order effects in the analysis.

Furthermore, the Eurocode 3 results to be one of the first standards to introduces the concept of imperfections; as it will be seen during the calculation examples in the design handbook, these can be "frame imperfections", "member imperfections" and the "imperfections for the analysis of the bracing system" (that are not relevant in this project).

We can now examine more deeply some themes regarding "sway frame" analysis:

- **Sway-non sway criteria (see section 5.2.5.2)**

In general, the sensitivity of a steel frame to the second order effects can be checked in the EC3 taking into account the value of the "elastic critical load ratio" (5.2.5.2(3)); a structure is "non-sway" if:

\[
\frac{V_{sd}}{V_{cr}} \leq 0.
\]

where:
\(V_{sd}\) = total design vertical load of the frame;
\(V_{cr}\) = elastic critical value of the total vertical load of the frame.
Vice versa the frame is considered as "sway".

For 2D-frames, the Eurocode also indicates a simpler rule (5.2.5.2(4)): this methodology can be applied starting from the results of an usual first order analysis. A structure is defined as "non-sway" if, for each storey of the frame, the following condition is fulfilled:

\[
\frac{\delta_i \cdot \Sigma V_i}{h_i \cdot \Sigma H_i} \leq 0.
\]

where:
- \( \Sigma V_i \) = is the total vertical reaction at the bottom of each storey;
- \( \Sigma H_i \) = is the total horizontal reaction at the bottom of each storey;
- \( \delta_i \) = is the horizontal displacement at the top of the storey relatively to the bottom determined from a first order analysis;
- \( h_i \) = is the storey height.

If only one storey fails to fulfil the criterion, the frame is considered as "sway".

- **Definition of "sway moments" (clause 5.2.6.2(5))**

According to the EC3, the sway moments are those associated with the horizontal translation of the top of a storey in relation to the bottom of the storey. These moments arise from the horizontal loads and also, in the case of asymmetrical structures and/or asymmetrical vertical loading, from the vertical loads.

- **Sway mode buckling lengths method (clause 5.2.6.2(8))**

This is one of the two methodologies included in the Eurocode that enables, in the analysis of "sway frames", to take into account indirectly the second order effects.

In the design of beams the axial forces \( N \) and the shear forces \( V \) are taken directly from the first order analysis. For the moments, instead, the values of the first order "sway moments" have to be incremented by a 1.2 factor, and thus the design values are calculated with:

\[
M_i = 1.2 \left[ M_i(H) + M_i^{SW}(V) \right] + M_i^{BD}(V)
\]

where:
- \( M_i(H) \) = bending moments due to the horizontal loads;
\( M^{iH} (V) \) = bending moments due to the part of the vertical loads related to the horizontal translation of the storeys;

\( M^{iBD} (V) \) = bending moments due to the part of the vertical loads related to the vertical deflection of the beams.

In the design of columns the values of M, N, and V are taken directly from the first order analysis. For the column buckling checks, effective lengths "Ls" for the sway mode shall be taken.

- Amplified sway moments method (clause 5.2.6.2(3))

In member design the axial forces N and the shear forces V are taken directly from the first order analysis. For the moments, instead, the values of the first order "sway moments" have to be incremented by a storey(i) "\( \alpha_i \)" factor given by:

\[
\alpha_i = \frac{1}{1 - \frac{\delta_i \cdot \Sigma V_i}{h_i \cdot \Sigma H_i}}
\]

The design value of the bending moments can then be determined with:

\[
M_i = \alpha_i \cdot \left[ M_i^H + M_i^{sw} (V) \right] + M_i^{BD} (V)
\]

For column buckling checks, effective lengths "Ls" for the non sway mode shall be taken.

2) Classification of cross sections (section 5.3)

The classification of cross sections is certainly one of the main aspects that differentiate the EC3 from some of the other National Standards. The classification is based on the capacity of a member of forming plastic hinges with sufficient rotation capacity. This possibility is largely conditioned by the influence of local buckling.

In practice, the classification is based on the value of a number of ratios depending on the geometrical characteristics of those cross section elements (webs, flanges) that result to be totally or partially in compression. According to these values, the cross section is classified from 1 to 4; in decreasing order are the cross sections with lower post-elastic capacities. In the design handbook is illustrated the operative sequence for this classification.
3) **Resistance and buckling checks of cross sections and members (sections 5.4 - 5.5 - 5.6)**

The calculation of the cross sectional resistances (e.g. compression resistance, bending resistance, bending and axial force resistance) and the buckling resistances of the member (e.g. axial, lateral-torsional) depend entirely on the classification of sections mentioned at point 2).

Furthermore, the buckling checks depend entirely on the buckling length "$L_b$" of the member, which depends on the restraint condition of the element. In "sway frames" "$L_b$" is greater than the storey height while in "non sway" frames, due to the lateral restraint of the frame, "$L_b$" is smaller than the storey height.

In fig. 2.1, the buckling lengths of "sway" and "non sway" frames are clearly illustrated.

**2.1.4 - The Connections**

In *chapter 6 of the EC3* are given the principles for the design of bolted and welded connections. For certain types of connections (beam-column end plate joints) there are specific annexes of the code with specific rules.

In the field of rigid frames only certain types of connections are important; it is certain that the most common type of joint in structures without bracing system is the end plate kind of connection; for this type of structures, complete and specific design rules are given in *Annex J.*
Column buckling lengths in "Sway" frames and in "Non Sway" frames

\[ L_b > h \]

SWAY FRAME

\[ L_b = h \]

NON SWAY FRAME

ECSC Project 7210-SA/419
Practical application of Eurocode 3 to multi-storey steel "Sway Frame" structures

FIGURE 2.1
2.2 - EUROCODE 1

In this work, the design loads have been taken from the Eurocode 1; the standards is still in a "draft" status but the current version can be considered to be quite "stable".

As it will be seen in the design handbook, the EC1 is divided in sections, one for each type of load. The rules are exhaustive and closely linked to the ones given in the specific National Standards.

2.3 - EUROCODE 8

The code has been divided in three parts; the first two are general while the third has specific rules for each different type of structural material. These are:

A) PART 1.1 - Seismic actions and general requirements for structures;
B) PART 1.2 - General rules for buildings;
C) PART 1.3 - Specific rules for buildings of different materials.

2.3.1 - Part 1.1

1) General

This part of the code contains general rules such as the requisites for the SLS and ULS checks in a seismic analysis, the definition of the seismic actions, the types of seismic analysis and the combination of the seismic effects with the other loads.

In particular, it is interesting to examine some of these points of relevance for this study:

2) Definition of the Limit States

With reference to the ULS, the structure shall be designed and constructed to withstand the design seismic action defined in the Eurocode without general or local collapse, thus retaining the structural integrity and a residual load bearing capacity after the seismic event (clause 2.1.1).

With reference to the SLS, the structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the cost of the structure itself.
It should be clear that the Eurocode refers to the ULS when determining the seismic action. When, instead, a SLS check is performed, it is necessary to define a number of parameters that enable to define a seismic action with a lower intensity.

2) Definition of the behaviour factor "q"

In order to take into account the energy dissipation capacity of the structure and to avoid to perform elasto-plastic analysis, the "behaviour factor" q has been introduced (see clause 2.2.1). The factor is equal to the ratio between the seismic intensity of an earthquake that leads the structure to collapse (taking into account its post-elastic capacity) and the design seismic intensity referring to a conventional linear model of the structure. It is thus evident that the higher is "q" the greater are the post-elastic capacities of the structure.

2.3.2 - Part 1.2

In this part, the concept of "structural regularity" is described together with its importance in the determination of the seismic actions and on the seismic design methods. In addition, some of the rules given in part 1.1 are specified for the case of multi-storey buildings.

It is necessary to remember that the EC8 enables the designer to adopt simplified methods for the seismic analysis only if the building can be classified as regular. The concept of structural regularity refers to the planar and elevation distribution of the structural elements in the building; depending on these configurations, the structure can have weak zones and thus may need to be considered as "non-regular".

2.3.3 - Part 1.3

In the last part of the EC8, specific rules are given to the different materials adopted in the seismic buildings. For steel structures a number of important clauses can be outlined:

1) Values of the behaviour factors "q"

The Eurocode gives the values of "q" for various types of steel structures: (see figure 3.1) these have higher values in the case of structures in which the dissipative capacities are of the flexural type while those with a shear or axial mechanism have much lower values (as an example it is sufficient to compare the value of "q" of a rigid frames with a braced frames).
Furthermore, according to clause 3.5.2.1, for the elements of the members in compression it is necessary to fulfil rules linking the value of the "q" factor with a minimum allowable cross-sectional class for the structures profiles.

2) Rigid frames (moment resisting frames)

For this category, two specific criteria shall be fulfilled (see point 3.5.4.1):

a) moment resisting frames shall be designed so that the plastic hinges form in the beams and not in the columns. This requirement not necessarily needs to be fulfilled at the base of the frame, at the top floor of multi-storey buildings and for one storey buildings;

b) the beam to column joints shall have adequate overstrength to allow the plastic hinges to be formed in the beams and not in the connections themselves.

2.4 - ECCS PUBLICATION No.33

The ECCS publication No.33 "Ultimate limit state calculation of sway frames with rigid joints" has been developed with the aims of:

• to making the results of different computer programs following the plastic zone theory more comparable by the use of the same important basic input parameters upon which members of 11 European countries have agreed unanimously;

• to show up for the ultimate limit state calculation of "sway frames" different possibilities of simplifications, or approximations, and the range of application.

The document main field of application is the analysis of frames with "I" or "H" sections loaded in the plane of their maximum bending stiffness.

2.4.1 - Definitions

1) Sway frames

According to the publication, the following structures are classified as "sway frames":

• the unbraced frames;
• the braced frames whose stiffness of the bracing system is relatively small; a practical criterion is given to verify this property:

\[ S_b < 5 \cdot S_u \]

where:
- \( S_b \) = stiffness of the bracing system;
- \( S_u \) = stiffness of the frame without the bracing system.

2) Rigid joints

These are:

• the welded connections;
• slip resistant connections with high grade bolts;
• other connections in which the deflections and the rotations between the connected members are negligible.

2.4.2 The analysis methods

The methods proposed by the publication No.33 are, starting from the most accurate and complex one down to the most simple:

1) Ultimate strength theory [plastic-zone theory] (section 1.1)

The method is based on an elasto-plastic analysis in which a stable equilibrium of the structure is found. The procedure requires the adoption of computer codes and, due to its complexity, is very little applicable by the designers and thus is adopted for research purposes and for the verification of the simplified methods.

2) Second order plastic hinge theory (section 1.2)

This is a more practical design method since, in comparison to method 1), it does not take into account the strain-hardening of the material and supposes that the post-elastic strains are concentrated in the "plastic hinges".

3) First order plastic hinge theory (section 1.3)
The method is based on a check regarding the values of the shear forces: when the additional storey shear forces, due to the axial forces in the columns and the horizontal displacements, are less than 1/10 of the storey shear it is sufficient to perform a first order analysis.

4) Second order elastic theory (section 2.1)

The method is very popular in common design situations. It takes into account the P-Δ effects with no iterations. Further details are given in clause 7.2.1 - prop. b).

5) First order elastic theory (section 2.2)

Similarly to method 3), the method is based on a check regarding the values of the shear forces: when the additional storey shear forces, due to the axial forces in the columns and the horizontal displacements, are less than 1/10 of the storey shear it is sufficient to perform a first order analysis.

6) Merchant-Rankine approach (section 3)

The method can be adopted for hand calculations; it is based on the calculation of the critical factor of the structure and it is used mainly in the UK.
3 - THE NATIONAL STANDARDS

3.1 - GENERAL

In this work, to better understand the different approaches for "sway frame" calculation in different European and Non European countries, a thorough study of the clauses of some National Standards regarding frame design has been performed.

It is evident that the degree of detail is very different form one code to the other; in fact some of these standards neglect many aspects involving second order calculations.

From this point of view, as soon as the EC3 will reach the "EN" status, the European code will enable a common approach among all the EEC countries in the calculation of these structures.

The codes that have been considered are:

- United Kingdom - BS 5950: Part1 - "Structural use of Steelwork in building" - 1990;
- France - "Règles de calcul des constructions en acier" - December 1966;
- Italy - CNR-UNI 10011 - "Costruzioni in acciaio - Istruzioni per il calcolo, l'esecuzione, il collaudo e la manutenzione" - June 1988;
- Germany - DIN 18800 parts 1&2 - "Structural Steelwork" - November 1990;
- Switzerland - SIA 161 - "Constructions métalliques" - 1990;
- USA - AISC - LRFD - "Specification for structural steel buildings" - September 1986;

3.2 - UNITED KINGDOM

3.2.1 - General

The general philosophy (i.e. the definition of "sway frames") of the BS 5950 is very similar to the Eurocode 3. The standard contains exhaustive rules for most of the specific problems regarding the design of common steel structures. It is in fact certain that many of the rules that have been included in the EC3 derive from the British code.

In the following sections only the main clauses regarding the analysis of "sway frames" will be described.
3.2.2 - "Sway-Non Sway" criterion

According to the code, a structure may be classified as "non sway" (see clause 5.1.3) if its horizontal sway is such that the secondary moments due to the non verticality of the columns can be neglected.

The standard gives a practical criterion valid for rigid multi-storey frames, that enables to check this requirement:

If a first order analysis is performed loading the frame with the storey "notional horizontal loads" given by:

\[ F_{nh,i} = \frac{p_{v,i} + q_{v,i}}{200} \]

where:
\( F_{nh,i} \) = horizontal notional load for storey \((i)\);
\( p_{v,i} \) = factored dead load of storey \((i)\);
\( q_{v,i} \) = factored vertical imposed load of storey \((i)\).

Under these forces, a frame is considered as "non sway" if, for every storey:

\[ \delta_i \leq \frac{h_i}{2000} \]

For cladded frames where the stiffening effect of the cladding is not taken into account in the deflection calculations

\[ \delta_i \leq \frac{h_i}{4000} \]

For cladded frames where the stiffening effect of the cladding is taken into account in the deflection calculations

where:
\( h_i \) = storey\((i)\) height;
\( \delta_i \) = storey\((i)\) horizontal deflection (of the storey top in relation to the storey bottom).

It is essential to observe that according to the BS the "sway-non sway" criterion is performed according to a well defined load case; in other codes, as it will be seen in the next pages, the load case is not specified and thus the structure may result to be "sway" for a number of load cases and "non sway" for others.
3.2.3 - Calculation methods for "sway frames"

According to the BS 5950, if a frame has been classified as "sway", then one of the following simplified methods (see section 5.6) should be adopted for the calculation of the structure:

1) Extended simple design

This is similar to the "sway mode buckling length" method of the EC3. In column design the internal forces deriving from a first order analysis shall be taken directly and the effective (buckling) length of the column obtained from appendix E allowing for side sway shall be adopted in column local checks.

The diagrams given in appendix E are the "buckling length" diagrams due to Wood; these enable to compute the effective length taking into account the geometrical and static characteristics of the beams and columns contiguous to the examined column.

2) Amplified sway method

This is similar to the "amplified sway moments" methods given in the EC3. The first order moments due to horizontal loading shall be amplified by the factor:

\[ \alpha = \frac{\lambda_{cr}}{(\lambda_{cr} - 1)} \]

where:
\[ \lambda_{cr} \] is the "elastic critical load factor" of the frame determined via the formulas provided in appendix F and given below.

\[ \lambda_{cr} = \frac{1}{200 \cdot \phi_{s,\text{MAX}}} \]

where:
\[ \phi_s = \frac{\delta_u - \delta_l}{h} \]

where:
\[ h \] = storey height;
\[ \delta_u \] = horizontal deflection of storey top due to notional horizontal loading;
\[ \delta_l \] = horizontal deflection of storey bottom due to notional horizontal loading;
\[ \phi_{s,\text{MAX}} \] = is taken as the maximum value among the values of "\( \phi_s \)" computed for each storey.
For the column buckling checks the effective lengths shall be taken as
\[ L_e = 1.0 \cdot L \]

where:
\[ L \text{ = storey height} \]

It is evident that rigorous second order calculations can also be performed instead of the two indirect methods given above.

3.3 - FRANCE

3.3.1 General

The French standard does not give, for usual buildings, any indication on the methodology to follow in the analysis of "sway frames". In the case of structures with "non usual" dimensions, instead, the code suggests (see clause 3.9) that when the following condition is not fulfilled:
\[ \delta < \frac{H}{200} \]

where:
\[ H \text{ = frame total height; } \]
\[ \delta \text{ = frame horizontal deflection. } \]

it might be necessary to check the global stability of the structure.

The above criteria is in fact a limit on the horizontal translation of the frame and not a check that enables to understand the sway behaviour of the structure; in fact, the criterion is not linked to the forces applied to the frame.

3.3.2 The "Additif 80"

The "additif" contains complementary rules to those specified by the standard (see section 7). The importance of the geometrical second order effects, according to the document, depends on the value of the factor "\( \alpha \)" that is the "elastic critical load factor" of the structure. In the "additif" the following cases are discussed:

\[ \leq \alpha < 5 \]

a second order analysis must be performed together with a check of the columns global rotation;
either a second order analysis (direct or indirect) or a first order analysis are permitted.

In this second case, if a first order analysis is performed it is necessary to perform some additional checks regarding also the value of \( \alpha_p \) factor (corresponding to the formation of a mechanism) and the columns' global rotation.

It is important to observe that no clear separation regarding the values of \( \alpha_{cr} \) has been introduced to define the structures in which the second order effects have to be taken into account and those in which a first order analysis is sufficient.

The document is clear in saying that higher the value of \( \alpha_{cr} \) and less is the structure sensitive to the second order effects. Nevertheless, there are no suggested simplified methods for the calculation of the coefficient and thus, in practice, the "additif" is applicable only if a rigorous second order analysis of the frame is performed.

3.4 - ITALY

3.4.1 General

Similarly to the French standard, the CNR 10011 does not have a complete set of rules for the analysis of "sway frames". The general lines of the code are not very recent, and thus do not go very deep into the clauses regarding the second order effects.

3.4.2 "Sway-Non Sway" criterion

According to the Italian code (see clause 7.5.2.3.1), the "lateral stability" criteria is based on the check of the frame total horizontal deflection \( \delta \) obtained by first order analysis with the following conventional loads:

\[
F_{h,i} = \frac{p_{v,i} + q_{v,i}}{80}
\]

where:
- \( F_{h,i} \) = horizontal conventional load of storey(i);
- \( p_{v,i} \) = dead loads of storey(i);
- \( q_{v,i} \) = imposed loads of storey(i).

A frame fulfils the "lateral stability" criteria if the following condition is satisfied:

\[
\delta \leq \frac{H}{500}
\]
where:
\( \delta \) = frame total horizontal deflection;
\( H \) = frame total height.

The interpretation of this rule is in fact a "sway-non sway" criterion; if the formula is verified, it is sufficient to perform an ordinary first order analysis of the frame; on the other hand, it is necessary to take into account the additional effects due to horizontal nodal displacements performing a second order analysis.

Like the French code, the CNR does no include any design methodology for the simplified calculation of "sway frames" (indirect second order methods). References are made to the "Additional Shears" method, described in [35], that being a method whose solution is determined only after a number of iteration is only applicable through a computer program.

3.5 - GERMANY

3.5.1 - General

Similarly to the BS, the DIN 18800 contains exhaustive rules on most of the main aspects for the design of steel structures. The general philosophy of the code is similar to the EC3 and, as it will be described in the next sections, the rules concerning "sway frames" are very similar.

Most of the formulas of interest for second order calculations seem to derive from [18].

3.5.2 - "Sway-Non Sway" criteria

- According to the German standard, a first order analysis is sufficient if the following condition is fulfilled by all the storeys of the frame (see clause 5.3.2.1):

\[
\eta_{k,r} = \frac{S_{r,d}}{1.2 \cdot N_r} \geq 10
\]

where:
\( N_r \) = sum of all vertical loads transmitted in storey(r);
\( S_{r,d} \) = frame stiffness of storey(r); the parameter depends on the geometrical and static properties of the frame and its members.
• If, instead, a first order analysis has been performed with given horizontal loads then the criteria can be alternatively written:

\[ \eta_{\text{tr}} = \frac{V_r^H}{\phi_r \cdot N_r} \geq 10 \]

where:
\( V_r^H \) = total shear of storey\((r)\); 
\( \phi_r \) = angle of rotation in storey\((r)\) due to a first order analysis with the given horizontal loads; 
\( N_r \) = as above.

### 3.5.3 - Elastic analysis of "sway frames"

The code specifies that if a first order analysis is performed and the condition given above is not fulfilled, then additional storey transverse forces shall be adopted.

• These incremented storey\((r)\) loads are given by (see 5.3.2.2):

\[ r = V_r^H + \phi_0 \cdot N_r + 1.2 \cdot \phi_r \cdot N_r \]

where:
\( V_r^H \) = total shear in storey\((r)\) due to the external horizontal loads; 
\( \phi_r \) = angle of rotation of storey\((r)\) due to the horizontal loads; 
\( \phi_0 \) = initial sway imperfection; 
\( N_r \) = total vertical loads transmitted within storey\((r)\).

• As an alternative method:

When:

\[ \eta_{\text{tr}} = \frac{S_{r,\text{d}}}{1.2 \cdot N_r} \geq 4 \]

the incremented storey horizontal force can be alternatively written as:

\[ V_r = \left( \frac{1}{\frac{1}{\eta_{\text{tr}}} - 1} \right) \left( V_r^H + \phi_0 \cdot N_r \right) \]
This last method is quite similar to the "sway moment amplified" method given in the Eurocode 3. Nevertheless, in both cases it is necessary to perform a new first order analysis since the design loads have been incremented and thus result to be different.

3.6 - SWITZERLAND

The code clearly (see section 3.251) states that when the structural deflections have an important influence on the internal forces of the members, a second order theory is essential.

On the other hand, no practical criteria are given to perform the check and no methods are specified for "sway frame" analysis.

3.7 - USA

The LRFD code has very limited indications and design rules regarding "sway frames" and second order calculations; In chapter C the American code specifies that the major difference between braced and unbraced frames is the value of the buckling factor "K" for the column. In the case of braced frames, "K" shall be assumed not more than unity, while in the unbraced frames its value is higher than one.

The standard indicates, for a certain number of different end support conditions, the values for "K" to be taken in the checks of the columns.

With the value of the buckling lengths it is possible to follow a similar approach to the "sway mode buckling length" of the Eurocode 3 in which the second order effects are taken into account indirectly by increasing the buckling length of the columns (with respect to the buckling lengths of a laterally restrained frame).

3.8 - JAPAN

The Japanese code, similarly to the LRFD, has very limited indications and design rules regarding "sway frames" and second order calculations; in clause 11.5 the standard specifies that the major difference between braced and unbraced frames is the value of the buckling factor "K" for the column. In the case of braced frames, "K" shall be assumed not more than unity, while in the unbraced frames its value is higher than one.
In table 11.3.1, the standard gives the values for "K" to be taken in the checks of the columns for a certain number of different end support combinations.

As the American code, the AIJ does not supply a practical criteria for the definition of a frame as "sway" or "non sway". In addition, no other methods regarding the increment of the horizontal forces have been introduced.

3.9 - CONCLUDING REMARKS

The review of the European codes clearly indicates that the general philosophy is similar as to the design of frames: the need of taking into account the importance of second order effects is recognised and in most cases rules are provided, which allow to check the frame sensitivity to geometrical P-Δ effects.

If due consideration is taken of the date of issue of the various National and European recommendations, it can be seen a sort of continuity from National Standards to Eurocode.

American and Japanese standards seem less sensitive to "sway frame" classification and design.
4 - THE CALCULATIONS ACCORDING TO THE EUROCODES

4.1 INTRODUCTION

Being the final aim of this work the preparation of the design handbook, the study approach was to apply to significant cases the EC3 rules in order to single out any operational problems.

A set of calculations have been performed on a number of forms of steel structures with expected "sway frame" behaviour, of interest amongst the major European countries. The selection was carried within the range of structures fulfilling specific structural requirements:

a) *Rigid beam-column connections frames* able to transmit bending, axial and shear effects, and thus carrying by themselves the lateral loads;

b) *Low rise frames* with not more than 5 storeys. In fact, in higher buildings it is certain that some type of bracing system is always present (steel bracing cantilevers or r.c. cores) since it would be otherwise very difficult for the frame to fulfil the SLS checks regarding the horizontal deflections.

It is important to add that the sensitivity of a frame to the second order effects is directly proportional to the entity of the vertical loads acting of the columns; consequently, the higher are the gravity loads in a given structure, the more easily shall the given structure behave as "sway".

The following four forms of buildings have been then chosen for the calculations:

- **MULTI-STOREY BUILDINGS FOR PARKINGS (PAR) - medium beam spans, low vertical loads**;

- **FRAMES FOR CIVIL BUILDINGS (CIV) - small beam spans, low vertical loads**;

- **WAREHOUSE BUILDINGS FOR MATERIAL STOCKING (WHS) - large beam spans, high vertical loads**;

- **INDUSTRIAL BUILDINGS WITH BRIDGE CRANES (IND) - large beam spans, low vertical loads**.

The details regarding the geometry of the "PAR" and "CIV" buildings, together with the structural models adopted for analysis, the assumed support conditions of the columns and the necessary hypothesis regarding the structural behaviour are described in the design handbook.
The "IND" building does not belong to the category of multi-storey frames but has been considered since it results to be a common structural solution in the case of industrial buildings with small-medium bridge cranes.

For each different structure, a seismic and a non seismic calculation analysis was performed; in particular, in the non seismic calculations, the methods of analysis indicated in section 5.2.6 of EC3 have been performed and the different sets of results have been compared. In this way it was possible to inspect the good features and the limits of each one of these methods.

In the seismic calculations, instead, the rules for the "simplified modal analysis" (EC8 - part 1.2 - section 3.3) have been followed, together with the clauses for the definition of the behaviour of the frame regarding the second order effects (EC8 - part 1.2 - clause 4.1.1.2). In this case, being these rules quite inadequate for the correct definition of simplified second order calculation methods, it was decided to extend to the seismic calculations the simplified methods given in section 5.6 of the EC3.

4.2 - THE OPERATIONAL PHASES

4.2.1 - General

A second order analysis requires sophisticated second order codes that are not yet available to a large number of designers and that are generally more complex to use. In addition these programs may require powerful hardware platforms (e.g. workstations or even mainframes) and are quite expensive if their cost is compared to that of a first order structural analysis program.

In any case, taken the scope of the work, a number of second order programs have been examined.

Amongst the identified second order analysis codes, (a complete list of these is included in the design handbook of "sway frame" steel structures, together with the main characteristics of each one of these) P.E.P. micro, developed by the French company C.T.I.C.M., was selected to perform the second order calculations in this phase.

Compared to some of the other second order codes, P.E.P. micro is simple and does not require very sophisticated hardware configurations.

The program leads the structure to the collapse condition by incrementing, step by step, the value of a "amplification load factor" that is applied to all the design loads.

As the load factor increases the determinant of the tangential stiffness matrix is reduced to zero. This limit leads to an indetermination of the displacement increment and a state of "instability" is reached. In any case it would have been preferable if the program would have kept the value of the horizontal loads constant and incremented only the vertical loads until the collapse condition was reached.
Nevertheless, in normal design practice this difference does not influence very much the elastic collapse condition and thus the value of the "elastic critical load factor".

With reference to first order analysis codes the number of the available programs is obviously much greater; these are the codes that are familiar to civil engineers and that normally do not require sophisticated computers. With these programs it is only possible to take into account the second order effects indirectly using one of the simplified methods included in clause 5.2.6.2 of EC3.

Due to the great number of frames to be analysed, it was important to use a program with an efficient pre-processor. For this purpose, STRANGER (Structural analysis generator), developed by the Italian company SIDERCAD in the context of the Eureka EU 130 Cimsteel Project was selected amongst the codes available for first order calculations.

In the next section is presented a brief summary of the design assumptions made during these calculation examples.

4.2.2 - Calculation assumptions

In every example of this calculation phase, a number of general assumptions and simplifications have been made for the determination of the structural models to be adopted in the analysis.

These are:

- **Influence of shear strains** - these have been omitted since, in usual calculations, do not influence the results;

- **Two dimensional frames** - every class of building has been divided in the structural analysis in two sets of simplified two dimensional frames. The first group, regarding the frames oriented in the direction of the smaller side of the building (transversal frames), concerns the frames with no bracing system (the lateral stability is guaranteed by rigid full strength connections) and a "sway" behaviour. The second set regards the longitudinal frames, of larger dimensions, that are provided with a cross bracing system. Our calculation examples regard only the first group of frames (transversal frames), being the second of no interest in this work (and already covered by Profil ARBED's project number SA/513);

- **Columns base support conditions** - in our working examples the columns are supposed either fully restrained or hinged. The second case is more common in relatively low buildings like those taken for our examples and gives structures with a higher degree of horizontal flexibility. The case of fully restrained supports for columns has also been considered, but it was evidenced that with this structural solution the horizontal flexibility was reduced and that the frame behaviour was very often "non sway";
• **Beam-column connections** - all the beam-column joints are supposed to be rigid full strength (i.e. full restraint) and capable to transmit the design axial forces, shears and moments. Semi-rigid connections have been neglected in this project since these are not included in the main objectives of the work and are not a popular solution in everyday practice;

• **Beam lateral support conditions** - in all our calculation examples we assume that the beams are laterally restrained and need not to be checked for lateral torsional buckling (*see EC3 5.5.2(8)*). This is acceptable in the case of multi-storey buildings, where the continuous r.c. slab (that we suppose non collaborating with the steel beams) gives a full lateral support condition and also for the industrial buildings where the lateral restraint is given by the purlins that are themselves braced by the steel roof sheeting;

• **Ground floors** - we suppose the presence of a loose stone foundation under the pavement of the ground floor. As a consequence the height of the first storey columns is 50-60 cm more than the remaining upper floors;

• **Non structural elements** - we suppose that these are fixed in such a way that they do not interfere with the frame deflections;

• **Structural spans** - these were selected taking into account the typical architectural characteristics and requirements of each form of building.

### 4.2.3 - Beam-column connection design

It has been said that all the calculation examples performed during this work have been performed on 2D-frames with rigid connections between the beams and the columns.

This type of joint (commonly known as "beam-column end plate connection") is normally obtained by welding an end plate to the end of the beam and subsequently connecting this plate to the column flange through bolts.

It is important to add that, if the joint stiffness is not sufficient, the column web may have stiffeners that normally are in line with the two flanges of the beam. In addition, if the shear resistance of the column is not sufficient, it can be necessary to weld a web plate to the column web in order to increase its shear resistance.

In the design of these connections, the rules given in EC3 - Annex J have been adopted. This part of the Eurocode is based on the determination of a number of "equivalent T-stubs" (*see EC3 section J.3.3*), representing each time a different part of the connection, used as models in the calculations.

The clauses regarding the design of these joints have not been included in the design handbook since these will soon be replaced by a completely revised document (annex JJ) with new design prescriptions; in addition this part of the design activities are independent from the "sway-non sway" behaviour of the structure.
5 - DESIGN CALCULATIONS ACCORDING TO THE NATIONAL STANDARDS

5.1 - GENERAL

Besides the calculations of a certain number of "sway frames" according to the EC3, some additional calculations have been performed taking some of the National standards examined in chapter 3.

As it has been evidenced before, one of the main topics regarding "sway frame" analysis results to be the classification of a frame according to its sensitivity to the second order effects. For this reason, special attention was given to the "sway-non sway" criteria of the different standards and some calculations were performed with these formulas.

The common structural model that was chosen was the multi-storey parking (PAR). This choice was due to the fact that being the frame symmetrical the vertical loads did not influence neither the determination of the "sway-non sway" criterion ratios nor the entity of the "sway moments" and thus the results of the analysis were obtained with a sufficient degree of accuracy to be able to perform efficient comparisons between the standards.

The National standards that have been considered are:

1) BS 5950 (GB);
2) CNR - UNI 10011 (I);
3) DIN 18800 - Part 2 (D);
4) Règles de calcul des constructions en acier (F).

The comparison regarded the computation of the "sway-non sway" ratios for the different storeys of the given frame according to the four standards taken into account and the subsequent comparison of these values between them.

The results indicated that certain codes are quite conservative (indicating to take into account the second order effects) while others specify that a first order analysis can be sufficient while according to the other codes this shows to be not true.

5.2 - UNITED KINGDOM

As it has been evidenced in 3.2.2, the classification of multi-storey frames as "sway" or "non sway" is very similar to the one given in the EC3 - clause 5.2.5.2(4).
According to the BS 5950 - section 5.1.3, the "sway-non sway" criteria is based on the check of each storey deflection \( \delta_i \) due to the "notional horizontal loads"; these horizontal actions are defined as:

\[
F_{nh,i} = \frac{P_{v,i} + q_{v,i}}{200}
\]

A frame is classified as "non sway" if, for every individual storey(i), the following condition is fulfilled:

\[
\delta_i \leq \frac{h_i}{2000}
\]

It is important to remember that the above given condition applies to the frames in which the effects of the cladding are not taken into account in the calculation of the structural deflections.

In our specific example (PAR), performing a first order analysis with the "notional horizontal" load case we obtain the following results regarding the horizontal "sway" of the storeys:

**Storey I**
\[
\delta_I = 0.3495 \text{ cm} > \frac{370}{2000} = 0.185 \text{ cm}
\]

**Storey II**
\[
\delta_{II} = 0.1356 \text{ cm} < \frac{310}{2000} = 0.155 \text{ cm}
\]

**Storey III**
\[
\delta_{III} = 0.0880 \text{ cm} < \frac{310}{2000} = 0.155 \text{ cm}
\]

**Storey IV**
\[
\delta_{IV} = 0.0938 \text{ cm} < \frac{310}{2000} = 0.155 \text{ cm}
\]
Storey V

\[ \delta_v = 0.0471 \text{ cm} < \frac{310}{2000} = 0.155 \text{ cm} \]

The frame is classified as "sway" according to the BS since the criteria is not fulfilled by the first storey.

The results of this "sway-non sway" check are very similar to those obtained with the check carried out according to the Eurocode 3 (see the Design Handbook - Annex II - Multi-storey parking non seismic calculation example, tables 2 & 3).

5.3 - ITALY

In our specific "PAR" example, taking the deflections due to the horizontal conventional loads introduced in 3.4.2, we obtain:

\[ \delta = 1.784 \text{ cm} < \frac{1610}{500} = 3.22 \text{ cm} \]

and thus a first order analysis is sufficient.

It is interesting to observe that according to the CNR the "sway-non sway" criteria is performed for the frame as a whole (taking the frame total height). Comparing the results to those obtained with the EC3 and the BS it is visible that the given approach results to be scarcely conservative since a first order analysis seems to be sufficient.

5.4 - GERMANY

As it has been described in 4.5.2, the rules given in the DIN 18800 part 2 - section 5.3.2.1 for "sway-non sway" classification are identical to those that appear in the ECCS publication No.33 (see point 2.1c)).

The values of the "sway-non sway" ratios are:

* For LOAD CASES 1 & 2

\[ \eta_{s,1} = 2.976 \quad (0.3360) \]

Storey II
In brackets (........) are given the inverses of the "sway-non sway" ratios that can be compared to those obtained with the EC3.

It is visible that according to the **DIN** the frame has to be classified as "sway" since the criterion fails for storeys **one, two and four**.

It is necessary to observe that this "sway-non sway" criterion results independent on the entity of the horizontal loads. In this specific example in which the vertical loads are the same from one load case to the other the values of the "sway-non sway" ratios refer to both load cases 1 & 2

**5.5 - FRANCE**

In the *Additif 80* the "sway-non sway" criterion is based on the value of "\( \alpha_{cr} \)" (see 3.3.2); in any case, there are no simplified expressions for the determination of "\( \alpha_{cr} \)" and thus the classification cannot be performed with the results of an ordinary first order analysis.

With reference to more rigorous calculations, from the second order analysis results of our "PAR" non seismic calculation the following value of "\( \alpha_{cr} \)" has been obtained:

\[
\alpha_{cr} = 4.000 \quad (0.2500)
\]
As a consequence, according to the French Standard (see 3.3.2) a second order analysis must be performed in this case, together with a check of the columns global rotation.

In table 5.1 it is possible to compare the frame "sway - non sway" classification results for the "PAR" example according to the National Standards taken in this review and the Eurocode 3.

<table>
<thead>
<tr>
<th>CODE</th>
<th>SWAY</th>
<th>NON SWAY</th>
</tr>
</thead>
<tbody>
<tr>
<td>EC3</td>
<td>⬤</td>
<td></td>
</tr>
<tr>
<td>BS (UK)</td>
<td>⬤</td>
<td></td>
</tr>
<tr>
<td>CNR (I)</td>
<td></td>
<td>⬤</td>
</tr>
<tr>
<td>DIN (D)</td>
<td>⬤</td>
<td></td>
</tr>
<tr>
<td>A.80 (F)</td>
<td>⬤</td>
<td></td>
</tr>
</tbody>
</table>

TABLE 5.1
6 - COLLABORATION BETWEEN PROJECTS SA/312, SA/419, SA/513

6.1 - GENERAL

As it has been mentioned previously, this project is closely linked to project SA/513 which covers all the frames (braced and not braced) that are classified as "non sway" according to the EC3.

The aim of the project is the draft of a practical design handbook for designers regarding the calculation of "usual (non sway) buildings"; as a consequence the two handbooks should be considered as complementary one to the other and enable to cover all the possible types of steel frameworks.

For "non sway" buildings further details can be obtained from the "Simplified version of the Eurocode 3 for usual buildings" design handbook of project SA/513.

6.2 - FLOW CHART FC1

The design phases to be followed for the two classes of frames have been indicated in a general flow chart of activities, named "FC1", that indicates, step by step, the possible design methods and the necessary design activities for each class of structure.

The diagram is the basis of the two works and has been developed in collaboration among CSM, CTICM and ProfilARBED recherches.

It is given in fig. 6.1

6.3 - COMPARISON BETWEEN A SWAY FRAME AND A NON SWAY FRAME

6.3.1 - General

In this work, a comparison between a rigid frame (with no additional bracing structures and a "sway" behaviour) and a braced frame (with a "non sway" behaviour) has been performed.

The two frames taken in this calculation are two-dimensional and have an identical geometrical layout; furthermore, the design loads to be taken in the calculation of the two structures are identical and thus the comparison can be considered to be carried out on two geometrically equivalent frames.

The braced structure that was taken in this comparison is frame WF2 from ProfilARBED's project SA/513 calculation example N.10 - P1-E1.

WF2 is one of the transversal bracing frames of the building; the beams and columns are H steel profiles, while the cross bracing members are double C sections. The connections between the beams and the column can be assumed to be pinned.
As it was specified before, the rigid frame has identical dimensions as WF2 and has beams and columns profiles that are H sections.

The models of the two structures are given in figs. 6.2 and 6.3

6.3.2 - Results

The "sway-non sway" classification of the EC3 has been checked for the rigid frame for 10 ULS design load cases; the frame resulted to be "sway" for 2 load cases and thus, for these two loadings, it was necessary to take into account the non linear geometrical effects performing a second order analysis. For the remaining ULS load cases, instead, it was sufficient to take directly the first order internal forces and moments.

For both frames the SLS checks (mainly for horizontal deflections) and the ULS checks (cross section and member checks) have been carried out and fully fulfilled.

The most interesting aspect of the comparison regards the total weight of the structural steel members (beams, columns, bracings). The values for the two structures are given below:

\[
\begin{align*}
\text{Rigid frame (sway)} & : 335 \text{ KN} \\
\text{Braced frame (non sway)} & : 202 \text{ KN}
\end{align*}
\]

As it would have been expected, the rigid frame results to be 65 % heavier than braced frame.

It is clear that in order to fulfil the SLS checks for the horizontal translation of the storeys it was necessary to strengthen the columns (adopting higher sections) and thus the weight of these vertical members results to be largely increased if compared to the braced frame.

On the other hand the braced solution brings severe limitations to the internal architectural layout of the building and a smaller degree of flexibility in its use. In addition, in seismic zones, it is interesting to observe that the EC8 enables (see part 1.3 - clause 3.5.1) to assume larger values for the behaviour factor "q" in rigid frames than in braced systems and thus the design seismic loads result to be lower in the first case.

Concluding, in buildings in non seismic zones it is evident that the cost of the steel structure can be largely reduced adopting some type of steel bracing system and thus leading the frame to a "non sway" behaviour. In other buildings, where the presence of the bracing structures can lead to severe restrictions in the use, the choice of a rigid frame may in many instances become advantageous, despite its higher cost.
Flow chart of Elastic global analysis of steel frames according to Eurocode 3 (Details)

1. Determination of load arrangements (EC1 and EC 8)
   - Load cases for ULS [2.3.3.1]
   - Pre-design of members, beams & columns ➔ Sections with pinned and/or rigid connections

2. ULS checks [Chap. 5]
3. SLS checks [Chap. 4]

4. Frame with bracing system
   - Design of the bracing system
     - Braced frame
       - Global imperfections of the frame
         - Non-sway frame [5.2.5.2]
           - $\delta_b \leq 0.2 \delta_0$
             - Sway frame
               - $V_{sl} / V_{et} \leq 0.1$
                 - $\lambda > 0.5 (A_{et} / N_{sl})^{0.5}$
                   - $\delta_0 < 0.1 \leq 0.25$
                     - $V_{sl} / V_{et}$
                       - $\lambda > 0.5 (A_{et} / N_{sl})^{0.5}$

5. Practical first order analysis
   - Non-sway mode buckling length approach
     - Sway mode buckling length approach
       - with sway moments amplified by factor $1/(1-V_{sl}/V_{et})$
         - $\delta_0 \leq 0.2 \delta_0$
           - $V_{sl} / V_{et} \leq 0.1$
             - $\lambda > 0.5 (A_{et} / N_{sl})^{0.5}$

6. Classification of the frame
   - Frame with bracing system ➔ Braced frame
     - Global imperfections of the frame
       - Non-sway frame [5.2.5.2]
         - $V_{sl} / V_{et} \leq 0.1$

7. Second order analysis
   - Members imperfections $\delta_{0d}$
     - $\delta_{0d} \leq 0.25$
       - $\delta_{0d} < 0.1$
         - $\lambda > 0.5 (A_{et} / N_{sl})^{0.5}$

8. Checks of the in-plane stability: members buckling (Chap. 5.3)
9. Checks of the out-of-plane stability: members and/or frame buckling (Chap. 5.5)
10. Checks of resistance of cross-sections (Chap. 5.4)
11. Checks of local effects (buckling and resistance of webs) (Chap. 5.6 and 5.7)
12. Checks of connections (Chap. 6 and Annex J)

FIGURE 6.1
"Sway" - "Non Sway" frames comparison
Braced "Non Sway" frame structural model and profiles

Note: the dimensions are indicated in millimetres

ECSC Project 7210-SA/419
Practical application of Eurocode 3
to multi-storey buildings with steel "Sway Frame" structures

FIGURE 6.2
"Sway" - "Non Sway" frames comparison
Rigid jointed "Sway" frame structural model and profiles

Note: the dimensions are indicated in millimetres

ECSC Project 7210-SA/419
Practical application of Eurocode 3
to multi-storey buildings with steel "Sway Frame" structures

FIGURE 6.3
7 - CONCLUSIONS

7.1 - GENERAL

With the appraisal of the prescriptions of the Eurocodes and their application to a selected number of frameworks, a number of comments regarding these design methodologies have been evidenced; the aim is twofold: inform the designer of steel frames with a "sway" behaviour on the correct interpretation and on the degree of reliability of each one of these rules and, secondly, advise the relevant technical committees of CEN of possible inconsistencies and even errors regarding practical rules included in the future European standards.

In this chapter the comments have been organised by subdividing them in a number of sections, each one correlated to a specific aspect of "sway frame" design in accordance with the Eurocodes.

7.2 - EUROCODE 3

The following points need to be critically reviewed:

7.2.1 The simplified "sway - non sway" criterion

- The sensitivity of a steel frame to the second order effects can be checked in the EC3 taking into account the value of the "elastic critical load ratio" given by:

\[
\frac{V_{sd}}{V_{cr}}
\]

where:

- \(V_{sd}\) = total design vertical load of the frame;
- \(V_{cr}\) = elastic critical value of the total vertical load of the frame.

In a given structure, the second order effects can be neglected if the value of the above ratio is \(\leq 0.1\) (see 5.2.5.2(3)). Otherwise, it is necessary to take into account this type of non linearity.

This "sway-non sway" criterion depends on how near is the design value of the vertical loads to the elastic critical condition; the closer are the two values, the greater is the need to take into account these second order effects.

The problems regarding the practical application of this formula are due to the fact that the determination of the exact elastic critical load is only possible via an eigenvalue analysis or the use of a second order computer code; it is sure that these features are not yet very common in computer programs utilised by designers in everyday design practice and thus this rules seems to be of scarce practical interest.
• The Eurocode also provides, at clause 5.2.5.2(4), a simplified rule that can be applied with the analysis results of a usual first order analysis; this recommendation requires that for each storey of the frame the following ratio is calculated:

\[ \frac{\delta_i \Sigma V_i}{h_i \Sigma H_i} \]

where:
\[ \Sigma V_i = \text{is the total vertical reaction at the bottom of each storey;} \]
\[ \Sigma H_i = \text{is the total horizontal reaction at the bottom of each storey;} \]
\[ \delta_i = \text{is the horizontal displacement at the top of the storey relatively to the bottom determined from a first order analysis;} \]
\[ h_i = \text{is the storey height.} \]

The second order effects can be neglected if the value of the ratio is \( \leq 0.1 \) for all the storeys. Vice versa, it is necessary to take into account this type of non linearity.
It is evident that the practical applicability of this simplified "sway-non sway" criterion is much greater since it is linked to common first order analysis results that are far more familiar to the engineers.

• It is important to outline that the two criteria described above are not associated with a particular loading case and thus should be checked for all loadings. As a consequence, the same structure can be "sway" for a certain number of load cases, while it may result "non sway" for others.

• As it has been pointed out in the previous chapters, a number of calculations have been carried out on frames whose column and beam layout resulted to be asymmetrical. These are the "CIV" and "WHS" buildings given in figs.7.1 and 7.2. The determination of the "sway-non sway" storey ratios (see 5.2.5.2 (4)) for these structures gave unexpected results: for different load cases, in which the entity of the vertical gravitational loads was kept constant, the values of the ratios for each storey resulted very different from one load case to the other. These results seem to contradict the principle that the sensitivity of a structure to the second order should depend only on the entity (and distribution) of the vertical loads and not on that of the horizontal forces.

In the first column of tables 7.1 and 7.2 (see EC3-HV) are given the values of these ratios for the two buildings. In table 7.3 are given the values of the "sway-non sway" storey ratios for the symmetrical "PAR" building given in fig.7.3.

As it has been previously specified, to storey ratios look incorrect for the "CIV" and "WHS" buildings while for the "PAR" building these seem to assume acceptable values.
- The inconsistencies pointed out in the previous paragraph for the analysis of asymmetrical frames suggest to find some alternative criteria, based on the results of an ordinary first order analysis, for the classification of frames.
Civil building (CIV) analysis
Frame structural model

Note: the dimensions are indicated in millimetres

ECSC Project 7210 SA419
Practical application of Eurocode 3 to multi-storey buildings with steel "Sway Frame" structures

FIGURE 7.1
Warehouse building (WHS) analysis
Frame structural model

Note: the dimensions are indicated in millimetres

ECSC Project 7210-SA/419
Practical application of Eurocode 3
to multi-storey buildings with steel
"Sway Frame" structures

FIGURE 7.2
Multistorey parking (PAR) analysis
Frame structural model

Note: the dimensions are indicated in millimetres

ECSC Project 7210-SA/419
Practical application of Eurocode 3
to multi-storey buildings with steel
"Sway Frame" structures

FIGURE 7.3
**COMPARISON BETWEEN THE STOREY ELASTIC CRITICAL LOAD RATIOS DETERMINED WITH DIFFERENT METHODS**

**calculation example:**

<table>
<thead>
<tr>
<th>Load case</th>
<th>Storey</th>
<th>Values of the load ratios according to the different methods</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td><strong>EC3 - HV</strong></td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>0.0687</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>---</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>0.1486</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.0229</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>0.2665</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.1502</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>0.1816</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.0766</td>
</tr>
</tbody>
</table>

**ANALYSIS METHODS**

- **EC3 - HV**: Load ratios determined in conformity to the EC3 - *clause 5.2.5.2(4)*
- **EC3 - H**: Load ratios determined in conformity to the proposed EC3 - *clause 5.2.5.2(4)* alternative formulation (see clause 7.2.1 - proposal a))
- **ECCS - P.33**: Load ratios determined in conformity to the ECCS publ. No.33- *clause 2.1(c)*
- **PEP**: Load ratios determined by a second order elastic analysis with "PEP Micro" (frame values)

**TABLE 7.1**
**ECSC SA/419 - “EC3 Sway Frames”**

**COMPARISON BETWEEN THE STOREY ELASTIC CRITICAL LOAD RATIOS DETERMINED WITH DIFFERENT METHODS**

**calculation example:**

**Warehouse building (WHS) - Non seismic**

<table>
<thead>
<tr>
<th>Load case</th>
<th>Storey</th>
<th>Values of the load ratios according to the different methods</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td><strong>EC3 - HV</strong></td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>0.1665</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.0773</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.0348</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>0.1296</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.0558</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.0285</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>0.0769</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.0272</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.0121</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>0.1126</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.0464</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.0237</td>
</tr>
</tbody>
</table>

**ANALYSIS METHODS**

- **EC3 - HV**: Load ratios determined in conformity to the EC3 - clause 5.2.5.2(4)
- **EC3 - H**: Load ratios determined in conformity to the proposed EC3 - clause 5.2.5.2(4) alternative formulation (see clause 7.2.1 - proposal a))
- **ECCS - P.33**: Load ratios determined in conformity to the ECCS publ. No.33- clause 2.1(c)
- **PEP**: Load ratios determined by a second order elastic analysis with "PEP Micro" (frame values)

**TABLE 7.2**
## COMPARISON BETWEEN THE STOREY ELASTIC CRITICAL LOAD RATIOS DETERMINED WITH DIFFERENT METHODS

### Calculation example:

**Multi-storey parking (PAR) - Non seismic**

<table>
<thead>
<tr>
<th>Load case</th>
<th>Storey</th>
<th>Values of the load ratios according to the different methods</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>EC3 - HV</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>0.2558</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.1142</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.0720</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.0820</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.0414</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>0.2548</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.1154</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.0735</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.0824</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.0434</td>
</tr>
</tbody>
</table>

### Analysis Methods

- **EC3 - HV**: Load ratios determined in conformity to the EC3 - clause 5.2.5.2(4)
- **EC3 - H**: Load ratios determined in conformity to the proposed EC3 - clause 5.2.5.2(4) alternative formulation (see clause 7.2.1 - proposal a))
- **ECCS - P.33**: Load ratios determined in conformity to the ECCS publ. No.33- clause 2.1(c)
- **PEP**: Load ratios determined by a second order elastic analysis with "PEP Micro" (frame values)

### Table 7.3
• PROPOSALS

a) In order to clarify the influence of the vertical loads on the values of the horizontal storey displacements (and thus of the "sway-non sway" ratios) a number of additional calculations of the "sway-non sway" ratio have been performed for the "CIV" and "WHS" frames using the modified version of formula 5.2.5.2(4) given below:

\[
\frac{\delta \cdot \Sigma V}{h \cdot \Sigma H} > 0.1
\]

in which we take:

\[\delta = \delta(H)\]

instead of the formulation given in the EC3:

\[\delta = \delta(H,V)\]

where:

\(H\) = horizontal forces (including equivalent forces due to frame imperfections);
\(V\) = vertical forces.

In the second column of tables 7.1, 7.2 and 7.3 are reported the values of the "sway-non sway" ratios for the alternative formulation with \(\delta = \delta(H)\) that can be compared with those obtained with \(\delta = \delta(H,V)\). It is apparent that for the analysis type "EC3-HV" the values of the above given ratios may be underestimated or overestimated depending on the direction of the horizontal forces. For "EC3-H", instead, the results seem to vary very little from one load case to the other and seem to estimate correctly the real behaviour of the structure.

It is necessary to emphasise that the calculation tests with this alternative formulation were limited to two different asymmetrical frames; even if the first results seem to indicate that this could be an alternative method, it seems necessary to perform other calculations in order to assess this new formulation.

b) In the E.C.C.S publication No.33 "Ultimate limit state calculation of sway frames with rigid joints" [18] a simple method is given for the determination of the storey "elastic critical load ratios" \(\eta_T\) (see 2.1 c)). The general formula is:

\[
\eta_T = \frac{5 \cdot (6 + k_r + k_{r-1})}{(2 + k_r) \cdot (2 + k_{r-1}) - 1} \cdot \frac{E \cdot C_r}{P \cdot h_r}
\]

where:
\[ k_r = \frac{C_r + C_{r+1}}{B_r} \]

\[ C_r = \frac{\Sigma I_{c,i}}{h_r} \]

\[ B_r = 2 \cdot \Sigma \frac{I_{b,i}}{I_{b,i}} \]

**FIGURE 7.4**

where:

- \( h_r \) = frame storey(r) height;
- \( L_{b,i} \) = frame bay(i) span;
- \( I_{c,j} \) = second moment of inertia of column(j) of storey(r);
- \( I_{b,i} \) = second moment of inertia of beam(i) of storey(r);
- \( P_r \) = total value of the design vertical loads at the bottom of storey(r).

It is apparent that the value of these coefficients depends only on the geometrical and static characteristics of beams and columns and on the entity of the vertical loads acting on the frame; this fact is coherent with the principle illustrated above regarding the fact that the sensitivity of a structure with regards to the second order effects depends on the vertical loads (and not on the horizontal).

On the other hand this approach would require a large amount of calculations and thus it is less suitable for routine design purposes.

In column 3 of tables 7.1, 7.2 and 7.3 may be noted the values of the storey ratios obtained with this method. It is visible that the method of the ECCS results more conservative than the one given in proposal a). Furthermore, in column 4 are given the values of the critical ratios determined by a rigorous second order analysis with the program "PEP-Micro". These values refer to the frame as a whole and thus should be compared with the higher storey value determined with one of the simplified methods described before.
* As an alternative, when an ordinary first order analysis with the horizontal loads "H_r" is performed, the values of the ratios can be calculated by:

\[
\eta_r = \frac{Q_r}{\psi_r \cdot P_r}
\]

where:
- \( H_r \) = total value of the design horizontal loads acting on storey(r);
- \( P_r \) = total value of the design vertical loads at storey(r) columns level;
- \( Q_r \) = total value of the design shear loads at storey(r) columns level;
- \( \psi_r \) = storey(r) columns inclination angle (due to a load case with only "H_r") determined from a first order analysis.

This formulation of the ECCS is identical to proposal a) and thus could result to be the most convenient alternative to the EC3 simplified criterion.

7.2.2 The definition of "sway moments"

- The concept of "sway moments" (see clause 5.2.6.2(5)) is of primary importance for the determination of the equivalent second order design bending moments according to the two indirect second order methods given in the EC3 at point 5.2.6.2(1).

- The definition of "sway moments" is not very clear in the case of asymmetrical structures and in the case of asymmetrical vertical loading; according to the EC3, all the moments that arise in such structures due to the action of the vertical loads have to be classified as "sway moments". If this is so, these indications seem to be incorrect: in general, the moments in a frame member are a sum of three different parts:

\[
M_i = M_i(H) + M_i^{sw}(V) + M_i^{BD}(V)
\]

* \( M_i(H) \) = part due to the horizontal loads;
* \( M_i^{sw}(V) \) = part due to the portion of the vertical loads related with the horizontal sway of the storeys;
* \( M_i^{BD}(V) \) = part due to the portion of the vertical loads related with the vertical deflection (primary bending) of the beams.

The problems is related to the determination of \( M_i^{sw}(V) \) and \( M_i^{BD}(V) \). The EC3 does not give any rule to determine the two components of \( M_i(V) \).

In order to make this definition to be applied in practice, it is hence necessary to introduce in section 5.2.6.2 of the Eurocode a criterion to enable the designer to
single out the part of the moments (due to the vertical loads) that are related to the translation of the storey, and the other part related to the primary bending of the beams.

**7.2.3 The amplified sway moments method**

- At point 5.2.6.2(3) the "amplification factor" seems to have a single value for the whole frame since it depends on the value of the "elastic critical load ratio". The formula given at point 5.2.6.2(6) equates a parameter that is fixed for the whole frame to a parameter that assumes different values from storey to storey. It is thus not clear if the "amplification factor" has to be taken as the highest among the storey ratios or if a different value should be assumed to each storey.

- Due to the uncertainties illustrated in 7.2.2 as to the determination of the "sway moments" in the case of asymmetrical frames, the method is not applicable for this type of structures. It seems as if it were necessary to find a new methodology for the indirect determination of the second order effects for these frames.

**Proposals**

a) As it has been already mentioned in 3.5.3, the DIN 18800 - part 2 indicates at clause 5.3.2.2 two simplified approaches to take into account indirectly the second order effects:

* the DIN standard defines a set of incremented storey(r) loads given by:

\[ V_r = V_r^H + \phi_o \cdot N_r + 1.2 \cdot \phi_r \cdot N_r \]

where:
- \( V_r^H \) = total shear in storey(r) due to the external horizontal loads;
- \( \phi_r \) = angle of rotation of storey(r) due to the horizontal loads;
- \( \phi_o \) = initial sway imperfection;
- \( N_r \) = total vertical loads transmitted within storey(r).

* As an alternative method, if the following condition is verified:

\[ \eta_{k,r} \geq 4 \]

the incremented storey horizontal force can be written as:
These two methods are more complex due to the fact that it is necessary to perform a new first order analysis with the incremented horizontal loads (while the "amplified sway moments" method is performed directly from the first order analysis results).

\[
V_r = \left( \frac{1}{1 - \frac{1}{\eta_{\text{cr}}}} \right) \cdot (V_r^* + \phi_0 \cdot N_r)
\]

b) In the E.C.C.S publication No.33 [18] is suggested a simplified second order calculation approach that is identical to the one given in the DIN Standard. The design storey(r) loads are multiplied by an amplification factor:

\[
Q_r = \left( \frac{1}{1 - \frac{1}{\eta_r}} \right) \cdot (H_r + \psi_0 \cdot P_r)
\]

where:

\[\eta_r = \text{storey(r) elastic load critical factor}\]

7.2.4 The sway mode buckling lengths method

- If for a given load case the "sway-non sway" ratio of point 5.2.5.2(4) has been calculated for all the storeys of the frame, it is advisable to adopt the "amplified sway moments" method as indirect second order method rather than the "sway mode buckling lengths" method. At least for symmetrical frames the first method results to be more accurate than the second, especially in the case of structures in which the sway index differs very much from one level to the other: in this case, while the "sway mode buckling lengths" method increases the "sway moments" by 20% in all the beams (independently of the actual sway behaviour), the "amplified sway moments" multiplies the "sway moments" by the corresponding storey "amplification factor" that differs from storey to storey.

In the columns, instead, the results of the "sway mode buckling lengths" method are quite similar to those obtained with the "amplified sway moments" method (see Design Handbook - Annex II - Non seismic civil (CIV) example, table 8 and Non seismic multi-storey parking (PAR) example, table 8).

- For the same reason as for the "amplified sway moments" method, the method results non applicable for asymmetrical frames.
7.2.5 Second order approach for SLS checks

- In chapter 4 (see 4.2.1(5)) the Eurocode specifies that the second order effects should be taken into account when calculating the serviceability deflections; nevertheless, there are no indications on how to obtain these second order displacements and, in addition, the limits given in 4.2.2(4) seem to refer only to deflections calculated in a first order analysis.

- It seems as if the rules of this part of the document were to be updated in order to enable the designer to take into account the second order effects in the SLS calculations in an efficient yet reliable way.

7.3 - EUROCODE 8

7.3.1 Second order approach for SLS checks

- The Eurocode 8 gives clear indications (see part 1.2 - clause 4.1.1.2) on whether the second order effects have to be taken into account or not. It is important to add that this criterion is equivalent to the one given in the EC3 (see 5.2.5.2(4)) for a non-seismic analysis.

- Nevertheless, the EC8 does not indicate any simplified method for the second order analysis. It would be desirable to include some design methodologies regarding this topic that is, instead, clearly treated in the Eurocode 3 (e.g. in section 5.2, the EC3 indicates the "amplified sway moments method" (5.2.6.2(3)) and the "sway mode buckling lengths method" (5.2.6.2(8))).

7.3.2 Values of the behaviour factor "q"

- The Eurocode 8 does not give any indication (see part 1.3 - clause 3.5.1) for the values of "q" (i.e. the behaviour factor) in the case of hinged column base columns. This case is very frequent for braced frames and it is also quite common in the case of rigid frames.
7.3.3 Post-elastic behaviour of frames

- According to clause 3.5.4.1 - part 1.3, the formation of plastic hinges should follow the following order: initially at the ends of the beams, while in the columns these should occur only in the base cross sections once all the beams ends have reached a plastic state.

- This post-elastic requirement for the frame requires a careful study of its plastic hinge formation and thus the use of unusual post elastic computer programs that are not very often at hand to small design offices. It would be thus desirable to add a more simple criterion to enable the designer to check on the post-elastic characteristics of the frame.

7.3.4 Horizontal deflection checks

- At clause 4.2.2 - part 1.2, the EC8 supplies the limits for the frame storey relative displacements to be fulfilled in a seismic analysis. Comparing these limits to the ones given in chapter 4 of the EC3 (clause 4.2.2(4)) it is evident that a check for the total frame deflections should be added in the EC8 as it appears in the EC3.

7.4 - GENERAL

7.4.1 Importance of the SLS horizontal deflection checks in "sway frames"

- The calculation examples of "sway frames" according to the Eurocode 3 pointed out that in the case of rigid frames the serviceability limit states checks regarding the horizontal frame deflections can be as important as the ultimate limit states verifications regarding member and cross-sectional checks. These preliminary SLS calculations are performed with appropriate load cases and are independent from the "sway-non sway" definition of the frame. In many cases it was necessary to strengthen the columns stiffness to reduce these horizontal displacements.

- In "sway frames", when design is governed by the control of deflections at the SLS, the incremented forces and moments due to second order effects have to be computed in any case since these are used in the design of the elements of the beam-column connections.
7.4.2 Influence of the shear strains

- Generally, the shear strains of frame members are neglected in design analysis. Checks on this assumption were carried out in the study: during the calculation examples it was evidenced that these strains influenced the results of the analysis and had to be taken as an extra design parameter.

- Due to this non negligible influence, three additional calculation examples have been performed taking into account these shear strains. These are:
  
a) Multi storey parking (PAR) - non seismic calculation;
  b) Civil building (CIV) - non seismic calculation;
  c) Warehouse building (WHS) - non seismic calculation.

In order to examine the influence of the shear strains in the design of "sway frames", a comparison of the values of the "sway-non sway" ratio (see EC3 - clause 5.2.5.2(4)) obtained during first order analysis was performed for the above mentioned analysis examples considering or not the shear strains. The ratios obtained with account for the shear strains are up to 15% greater than those obtained with no account for shear strains.

- On the other hand, the formulas given in section 5.2 of the EC3 for "sway frame" analysis should refer to calculations with no shear strains and thus their use in calculations with shear strains could result to be incorrect. In any case the Eurocode 3 should mention whether these effects have to be taken into account or not.

7.4.3 Influence of the column bases support conditions

- In the calculation examples of this work all the frames that have been taken were considered to have hinged column bases; other calculations, that have not been included, have been performed and indicated that a frame with the same profiles but with rigid joints at the column bases results to be far more stiff and are classified as "non sway".

- The main difference between the deflected configurations of the two types of support conditions regards the deformation of the first storey; with rigid column bases the horizontal deflections have small values, for hinged column bases these are much bigger and lead the first storey not to fulfil the "non sway" criterion.
8 - FURTHER RESEARCH

Further research and testing is required in order to revise thoroughly some of the clauses given in the Eurocode 3 - section 5.2 regarding "sway frame" design. In particular, the following points require further investigations:

(a) the development of a new simplified "sway-non sway" criterion to replace the approach given in clause 5.2.5.2(4);

(b) the set up of a practical criterion for the determination of the "sway moments" in an asymmetrical frame to replace the one given in 5.2.6.2(5);

(c) the development and validation of an improved indirect second order calculation method to replace the "moment amplification method" of clause 5.2.6.2(3).
## 9 - LIST OF SYMBOLS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIV</td>
<td>Civil building calculation example</td>
</tr>
<tr>
<td>CEN</td>
<td>European Committee for Standardisation</td>
</tr>
<tr>
<td>ECCS</td>
<td>European Convention for Constructional Steelwork</td>
</tr>
<tr>
<td>ECSC</td>
<td>European Coal and Steel Community</td>
</tr>
<tr>
<td>EC1</td>
<td>Eurocode 1</td>
</tr>
<tr>
<td>EC3</td>
<td>Eurocode 3</td>
</tr>
<tr>
<td>EC8</td>
<td>Eurocode 8</td>
</tr>
<tr>
<td>FC</td>
<td>Flow chart</td>
</tr>
<tr>
<td>IND</td>
<td>Industrial building with bridge crane calculation example</td>
</tr>
<tr>
<td>MAX</td>
<td>Maximum</td>
</tr>
<tr>
<td>MIN</td>
<td>Minimum</td>
</tr>
<tr>
<td>NAD</td>
<td>National Application Document</td>
</tr>
<tr>
<td>PAR</td>
<td>Multi-storey parking calculation example</td>
</tr>
<tr>
<td>SLS</td>
<td>Serviceability Limit States</td>
</tr>
<tr>
<td>3D</td>
<td>Three dimensional</td>
</tr>
<tr>
<td>2D</td>
<td>Two dimensional</td>
</tr>
<tr>
<td>ULS</td>
<td>Ultimate limit states</td>
</tr>
<tr>
<td>WHS</td>
<td>Warehouse building calculation example</td>
</tr>
</tbody>
</table>
10 - BIBLIOGRAPHY

1) "Eurocode 1 - Basis of design and actions on structures" (April 1993).


3) "Eurocode 8 - Structures in seismic zones - design" (May 1988).

4) "BS 5950 - Structural use of steelwork in building - Part 1" (1990).

5) "Règles de calcul des constructions en acier" (December 1986).


10) C. Poggi (Polit. di Milano) - R. Zandonini (Univ. di Trento) "A finite element for the analysis of semi-rigid frames" (1986).

11) U. Vogel (Univ. of Karlsruhe) "Alcuni commenti alla pubblicazione N. 33 della CECM: Calcolo allo stato limite ultimo di telai a nodi mobili con giunti rigidi" from "Costruzioni Metalliche" (1985).


13) F. Casciati (Univ. di Pavia) "Analisi elasto-plastica di telai piani a nodi spostabili in acciaio" from "Costruzioni Metalliche N. 1" (1980).


15) F.P. De Martino "Analisi mediante calcolo ad elementi finiti del comportamento di un telaio piano a nodi semirigidi" from "Acciaio" (May 1986).
16) T. Stelmac - M. J. Marley - K. H. Gerstle (Univ. of Colorado) "Analysis and tests of flexibly connected steel frames" from "Journal of structural engineering - Vol. 112" (May 1986)

17) K.M. Ang - G.A. Morris (Univ. of Manitoba) "Analysis of three dimensional frames with flexible beam-column connections" from "Canadian Journal of civil engineering - Vol. 11" (1984)

18) ECCS "Calcolo allo S.L.U. di telai a nodi mobili con giunti rigidi" ECCS - publication n.33 (1985)

19) Socrates - A. Ioannides (S.A.I.) "Frame analysis including semi-rigid connections and P-delta effects" (1986)

20) M.R. Horne (Univer. of Manchester) "Frame instability and the plastic design of rigid frames" (1980)


22) G. Ballio (Polit. Milano) - L. Calado (Univ. Lisbona) "Il ruolo della duttilità nella resistenza sismica delle strutture in acciaio" from "Acciaio" (September 1986)

23) E. Bozzo - L. Gambarotta "Inelastic analysis of steel frames for multistorey buildings" from "Computers and Structures Vol. 20 N. 4" (1985)


25) F.M. Mazzolani (Univer. di Napoli) "Influence of semi-rigid connections on the overall stabilità of steel frames" (1987)

26) G. Alpa - E. Bozzo - L. Gambarotta (Univ. di Genova) "Influenza della distribuzione delle rigidezze nella stabilità delle strutture a telaio per edifici multipiano" from "Costruzioni metalliche N. 6" (1982)

27) A. De Luca (Univ. Potenza) - C. Faella (Univ. Salerno) - V. Piluso (Univ. Salerno) "L'approccio dell'Eurocodice 3 per la verifica di sicurezza di telai a nodi spostabili" from "Atti del Collegio dei Tecnici dell'Acciaio - Abano Terme 1991" (1991)

28) P. Pilvin (ENSET-Cachan) "Modele de comportement statique des liaisons de structures a barres" (1983)

29) Y. Galea (C.T.I.C.M.) "Non-linear analysis of plane frame structures with semi-rigid connections" (1986)


33) E. Cosenza - A. De Luca - C. Faella "Stabilità in campo elastico ed inelastico di telai in acciaio a nodi semirigidi" from "Acciaio" (December 1986)

34) C.A. Castiglioni (Polit. Milano) "Strutture in acciaio contro il rischio sismico" from "Acciaio" (March 1987)

35) G. Ballio - M. Mazzolani "Strutture in acciaio" from page 484 to page 486 and from page 643 to page 669 - Ed. Ulrico Hoepli Milano


37) E. Bozzo - L. Gambarotta (Univ. Genova) "Sulla risposta sismica di edifici multipiano a struttura spaziale" from "Costruzioni Metalliche N.3" (1983)


39) G. Sedlacek (RWTH Aachen) - G. Ballio (Polit. Milano) "The response of steel structures with semi-rigid connections to seismic actions" (1987)

40) CNR - UNI 10011 "Costruzioni in acciaio - Istruzioni per il calcolo, l'esecuzione, il collaudo e la manutenzione" (June 1988)

41) AISC "Load and resistance factor design specification for structural steel buildings" September 1, (1986)

42) SIA "SIA 161 - Constructions métalliques" (1991)

43) DIN 4150 Part 1 - Vibration in building; Principles, predetermination and measurement of the amplitude of oscillation - English version
44) DIN 4150 Part 2 - Effects on humans in buildings - English version

45) DIN 18 800 Part 1 - Structural steelwork; design and construction - English version - (November 1990)

46) DIN 18 800 Part 2 - Structural steelwork; analysis of safety against buckling of linear members and frames - English version - (November 1990)

47) M.R. Horne (Univer. of Manchester) - "An approximate method for calculating the elastic critical loads of multi storey plane frames" from "The structural engineer N.6" (June 1975)

48) A. De Luca (Univ. Potenza) - C. Faella (Univ. Salerno) - V. Piluso (Univ. Napoli) "Stability of Sway Frames: different approaches round the world" (May 1991)

49) A. De Luca (Univ. Potenza) - C. Faella (Univ. Salerno) - E.Cosenza (Univ. Salerno) "Elastic buckling of semi-rigid sway frames" chap. 8 in "Structural connections: stability and strength" (1989)

50) A. De Luca (Univ. Potenza) - C. Faella (Univ. Salerno) - E.Cosenza (Univ. Salerno) "Inelastic buckling of semi-rigid sway frames" chap. 9 in "Structural connections: stability and strength" (1989)

51) A. De Luca (Univ. Potenza) - F.M. Mazzolani (Univ. Napoli) - V. Piluso (Univ. Napoli) "Stability of columns and beam columns: Eurocode 3 vs. other codes" (1991)

52) ECCS - TC5 "Software for Constructional Steelwork" (1990).
11 - LIST OF FIGURES AND TABLES

Figure 2.1  Column buckling lengths in "sway" frames and in "non sway" frames

Table 5.1  Comparison of the frame classification results for the "PAR" example according to some National Standards and the Eurocode 3.

Figure 6.1  Flow chart FC1: Elastic analysis of steel frames according to the Eurocode 3

Figure 6.2  "Sway-Non Sway" frames comparison - Braced "Non Sway" frame structural model and profiles

Figure 6.3  "Sway-Non Sway" frames comparison - Rigid jointed "Sway" frame structural model and profiles

Figure 7.1  Civil building (CIV) analysis - Frame structural model

Figure 7.2  Warehouse building (WHS) analysis - Frame structural model

Figure 7.3  Multi-storey parking (PAR) analysis - Frame structural model

Figure 7.4  ECCS publication No.33 - Scheme for the determination of the storey critical ratios

Table 7.1  Comparison between the storey elastic critical load ratios determined with different methods- CIV building

Table 7.2  Comparison between the storey elastic critical load ratios determined with different methods- WHS building

Table 7.3  Comparison between the storey elastic critical load ratios determined with different methods- PAR building
E.C.S.C. STEEL RESEARCH PROGRAM
Contract number 7210-SA/419

Practical application of Eurocode 3 to multi-storey buildings with steel "Sway Frame" structures

ANNEX 1

Design handbook for "sway" steel buildings design according to the Eurocode 3
INDEX

1 - INTRODUCTION TO THE DESIGN HANDBOOK ........................................ 72
   1.1 - AIMS .............................................. 72
   1.2 - HOW TO READ THIS HANDBOOK ........................................... 72

2 - SECTION A: PROCEDURES FOR THE DESIGN AND CHECK OF "SWAY FRAMES" ......................................................... 73
   2.1 - GENERAL .............................................. 73
   2.2 - PRINCIPLES .............................................. 73
   2.3 - NOTATIONS IN FLOW CHARTS .............................................. 74
   2.4 - THE FLOW CHARTS .............................................. 76
       2.4.1 - FC 1 .............................................. 76
       2.4.2 - FC R1.0 .............................................. 81
       2.4.3 - FC R1.1 .............................................. 81
       2.4.4 - FC R1.2 .............................................. 82
       2.4.5 - FC R1.3 .............................................. 82
       2.4.6 - FC R1.4 .............................................. 83
       2.4.7 - FC R1.5 .............................................. 83
       2.4.8 - FC R1.6 .............................................. 84
       2.4.9 - FC R1.7 .............................................. 84
       2.4.10 - FC R2 .............................................. 86
       2.4.11 - FC R3 .............................................. 86
       2.4.12 - FC R4 .............................................. 87
       2.4.13 - FC R8 .............................................. 88
       2.4.14 - R12.1 .............................................. 89
       2.4.15 - FC R12.2 .............................................. 90
       2.4.16 - FC R12.3 .............................................. 91
       2.4.17 - FC R16.0 .............................................. 92
       2.4.18 - FC R16.1 .............................................. 93
       2.4.19 - FC R16.2 .............................................. 94
       2.4.20 - FC R16.3 .............................................. 94
       2.4.21 - FC R16.4 .............................................. 95
       2.4.22 - FC R16.5 .............................................. 95
       2.4.23 - FC R16.6 .............................................. 95
       2.4.24 - FC R16.7 .............................................. 96
       2.4.25 - FC R16.8 .............................................. 96

3 - SECTION B: CALCULATION EXAMPLES ON SWAY FRAMES .......... 98
1 - INTRODUCTION TO THE DESIGN HANDBOOK

1.1 - AIMS

The present publication is intended to be a practical tool for the design, according to the Eurocode 3 (EC3), of a particular class of steel buildings in which the beam-column connections are full strength and the influence of the storey horizontal deflections needs to be taken into account for the correct determination of the internal forces and moments. According to the EC3, these structures are defined as "sway frames" (see section 5.2.5.2).

On the other hand, for the structures that behave as "non sway" is available ProfilARBED's project SA/513 publication "Design handbook according to Eurocode 3 for braced or non-sway steel buildings".

The present handbook gives all the indications for the design of "sway frames". Nevertheless it does not substitute in any case the original documents Eurocodes 1, 3 and 8.

The document is divided in three major sections:

- SECTION A - Procedures for the design and check, according to the Eurocodes, of steel structures with a "sway" behaviour;
- SECTION B - Calculation examples on "sway frames";
- SECTION C - Information on general purpose second order computer programs available in the European market.

1.2 - HOW TO READ THIS HANDBOOK

As it has been specified above, in Section A are illustrated the phases for the design of "sway frames". These procedures are given in a "flow chart" format and include all the clauses and design rules of the Eurocodes 1, 3 and 8 that are related to this type of structures.

The examples given in Section B are a practical application of the procedures given in Section A. The two parts should be read simultaneously in order enable a more efficient comprehension of the design rules.

As it will be seen further on more in detail, the practical examples are linked, step by step, to the design procedures.
2 - SECTION A: PROCEDURES FOR THE DESIGN AND CHECK OF "SWAY FRAMES"

2.1 - GENERAL

The design procedures are described with a number of flow charts: these are a sequence of boxes, with different shapes (according to the type of action that is being performed), containing instructions, messages, calculations, conditional statements, etc.

The flow charts are indicated in the same sequence as the designer should perform the different calculation phases.

2.2 - PRINCIPLES

The following principles have been adopted in the draft of the flow charts:

- A general flow chart regarding the elastic design of 2D-steel frames has been developed. It has been called FC 1 and has been developed in collaboration with CTICM and ProfilARBED during the development of the research projects SA/312 and SA/513;

- Being a general diagram for the elastic design of steel structures, FC 1 has also been included in ProfilARBED's publication "Design handbook according to Eurocode 3 for braced or non-sway steel buildings";

- In FC 1 all the design steps regarding the calculation of steel structures (i.e. from the load determination to the checks of the connections) are briefly indicated with corresponding "key words" but no details are given on each of these. FC 1 is a generic diagram that gives only a global view of the activities to be undertaken by the designer;

- The details on each design phase are given in other flow charts that indicate each single calculation related to one particular step;

- The formulas of the Eurocodes have not been re-written but only a reference to these equations has been introduced. When, instead, the needed formulas do not appear explicitly in the standards, these have been introduced in the boxes of the flow charts.

- In all the flow charts the design steps are numbered;
2.3 - NOTATIONS IN FLOW CHARTS

All the flow charts included in the design handbook should be read according to the following rules:

- the flow charts are read, in general, from the top to the bottom;
- the sequence of the "boxes" to be followed by the engineer is indicated by the arrows;
- the references to the relative rules of the Eurocodes are given in (....);
- the flow charts (other than FC 1) are numbered adopting the following convention:

\[ \text{FC } RX.Y \]

where:

\[ X = \text{row number of FC 1 containing the object of the given flow chart;} \]
\[ Y = \text{reference number of the flow chart. The following assumptions are made:} \]
\[ Y = 0 \quad \text{first level flow chart} \]
\[ Y = 1, 2, 3, ... \quad \text{second level flow chart} \]

It is thus evident that each flow chart (or groups of flow charts with the same value of "X") is a specification of the contents of a particular row of FC 1.

- the following convention for the box shapes is assumed:
- the intersection between two intersecting lines is effective only in the following case:

- the intersection between two intersecting lines is non effective when:
• when numerous branches start from the end of a vertical path (see figure below) the design steps related to each one of these branches have to be carried out.

2.4 - THE FLOW CHARTS

These are given in Annex I

2.4.1 - FC 1

♦ General

In FC 1 are illustrated six different vertical paths for the global analysis of steel frames according to the EC3; from path 1 to path 6 the calculation methods become gradually more sophisticated. In particular:

a) path 1 - consist in an ordinary first order analysis; the design indications of this method are given in ProfilARBED's publication "Design handbook according to the Eurocode 3 for braced or non-sway steel buildings".

b) paths 2-6 - these methods take into account either directly or indirectly the geometrical second order effects and are object of the design procedures contained in this handbook.

The analysis method given in path 6 should be avoided; it results to be over conservative and may lead to over dimensioning. In fact:

i) the designer performs a second order analysis in any case, even when a first order analysis results to be sufficient;

ii) the designer always introduces the member imperfections in the global analysis, even when these could not be necessary.

♦ Specific comments

* row 1:

• where:

EC1: Draft Eurocode 1 Basis of design and actions on structures.

EC8: Draft Eurocode 8 Earthquake resistant design of structures.

For further details see:

⇒ FC R1.0

⇒ FC R1.1, FC R1.2, FC R1.3, FC R1.4, FC R1.5, FC R1.6, FC R1.7

* row 2:

- where:

ULS: Ultimate limit states

SLS: Serviceability limit states

For further details see:

⇒ FC R2

* row 3:

- The design procedures illustrated in this design handbook are based on the assumptions on the type of connection between beams and columns: flow chart FC 1 covers only the case of perfectly pinned and full strength connections. No intermediate situations are covered in this work (i.e. semi-rigid connections).

For further details see:

⇒ FC R3
Both SLS and ULS checks have to be carried out for all the members of the steel frame.

For further details see:

\[ FC \ R4 \]
• The present "sway-non sway" criterion is based on the value of the "elastic critical load ratio":

\[ \frac{1}{\alpha_{cr} \cdot V_{cr}} = \frac{V_{sd}}{V_{cr}} \]

EC3 - 5.2.5.2(4) • For 2D-steel frames it is possible to adopt a simplified methodology for the "sway-non sway" check:

\[ \frac{1}{\alpha_{cr \cdot i \cdot v}} = \left( \frac{\delta_i}{h_i} \right) \left( \frac{\Sigma V_i}{\Sigma H_i} \right) \]

The difference between the two approaches is essentially due to the fact that the general method provides a value for the whole frame, while the specific method for 2D-frames gives a value to each storey of the frame.

In this second case "1 / αcr" shall be taken as the maximum value among the values of "1 / αcr,i":

\[ \frac{1}{\alpha_{cr}} = M_{AX} \left( \frac{1}{\alpha_{cr \cdot i \cdot v}} \right) \]

For further details see:

⇒ FC R8

* row 11

EC3 - 5.2.6.2(1) • In the case of "sway frames", the EC3 enables to adopt either first order methods with allowance to second order effects (paths 2, 3) or rigorous second order approaches (paths 4, 5, 6).

For further details see:

⇒ FC R12.1, FC R12.2, FC R12.3

* row 13
• As it has been already specified, path 6 results to be over conservative since neither the "sway-non sway" check (row 8) nor the check for "member imperfections" (row 9) are performed and thus the member imperfections have to be introduced in all columns.

* row 16

• As it will be evidenced further on, before the ULS member checks (buckling and resistance) are performed it is necessary to perform the "classification of cross sections" of all the frame members.

For further details see:

⇒ FC R16.0

⇒ FC R16.1, FC R16.2, FC R16.3, FC R16.4, FC R16.5, FC R16.6, FC R16.7, FC R16.8

* rows 17, 18, 19, 20, 21

• The sequence of these ULS checks is not imposed and it is up to the designer to choose the sequence to perform the verifications.

* row 21

• As it has been evidenced above, the assumptions made in row 3 regarding the stiffness of the beam-column connections have to be checked. For example, if a joint has been considered as a full strength (rigid) joint it is necessary to verify if it behaves really as a moment connection or if it has to be considered as semi-rigid.
2.4.2 - FC R1.0

♦ General

This flow chart specifies the global approach for the determination of each single load case. The flow charts recalled at row 6, instead, refer to each individual design load.

♦ Specific comments

* row 2:

- This step is essential for the determination of some types of actions; in fact, in addition to the structural elements, it is necessary to have information of the non structural elements. As an example, for the wind loads it is essential to know the characteristics and layout of the cladding elements.

* row 5:

- The loads that have been included are those that are normally of interest in the analysis of multi-storey frames. It is evident that other loads may have to be considered in particular design situations regarding multi-storey frames or in different types of structures like single storey portal frames.

- The frame imperfection loads have been considered "permanent actions" since these have been introduced to represent the effects of practical imperfections, that are a characteristic of the members of the structure. This assumption results to be conservative since part of the frame total vertical storey loads "Wi" (see FC R1.3, row 2) is due to the storey live loads.

2.4.3 - FC R1.1

* row 3, 4:

- In the structural analysis of the frame, the designer can decide whether to take into account or not the self weight of the joint elements (bolts, plates, cleats, welds, etc).
• To take into account these connection elements self weight a possible method is to increment the members self weight of a given percentage. For example, in the calculation examples performed during the research project SA/419, the profiles self weight has been increased by 10%.

2.4.4 - FC R1.2

* rows 2, 3, 4:

• Once the unit loads have been determined, it is necessary to refer to the loads acting on the frame. These can be only obtained if the portion of the deck referring to the 2D-frame is calculated. (e.g. for the pavement elements these are given in "KN/m2"; once the width of the building related to the frame (given in "m") is calculated, it is possible to determine the loads acting on the beams (given in KN/m))

2.4.5 - FC R1.3

* row 2:

• The value of "Wi" should be calculated taking into account all the gravitational loads:

\[ W_i = W_{iswL} + W_{iDL} + W_{iIL} \]

where:

- \( W_{iswL} \): vertical self weight loads
- \( W_{iDL} \): vertical dead loads
- \( W_{iIL} \): vertical imposed loads

* row 5:

EC3 - 5.2.4.3(8) • The additional storey loads "Fi" due to the frame imperfections should be taken into account only in the design of the steel members of the frames. In the determination of the horizontal reactions at the column bases (for the foundations design), instead, these additional actions should not be considered
2.4.6 - FC R1.4

* row 3

- The concentrated loads "Qk" have not been considered since these refer to specific design situations while this handbook indicated common rules of design practice.

* rows 3, 4, 5

- The following units refer to the design parameters:

  \( q_k \rightarrow \text{KN/m}^2 \)
  \( b \rightarrow \text{m} \)
  \( q \rightarrow \text{KN/m} \)

2.4.7 - FC R1.5

* row 2

- The value of "\( \mu \)" depends on:
  a) the pitch angle of the roof;
  b) the presence or not of the parapet in the roofs edge.

* rows 6, 7

- In current design situations it is necessary to compare the value of the snow load on roof "s" with the value of the imposed load "qk" determined in FC R1.4 - row 3. Taking into account that only one of the two loads can be present, the roof level of the building is loaded with the highest of the two vertical actions.

* rows 5, 8, 9

- The following units refer to the design parameters:
For the calculation of the wind pressure "we(z)" all the calculations of the four vertical paths between rows 2 and 4 have to be performed by the designer.

For the calculation of the design value of the spectrum $\beta(T_0)$ all the calculations of the five vertical paths between rows 2 and 4 have to be performed by the designer.

The value of the 2D-frame natural period "$T_0$" should be determined performing a rigorous dynamic analysis of the structural model. Nevertheless, if a simplified modal analysis is permitted (see row 6), the EC8 enables to estimate the value of the natural period with Rayleigh's formula:

$$T_0 = \frac{\sqrt{5}}{5}$$
where:

\[ \delta \] : total horizontal displacement (in cm) of the building subject to a horizontal loading equal to the gravitational loads

\[ T_0 \] : period obtained in seconds

- The value of the ground acceleration "a/g" is not specified by the Eurocode. The National authorities of each country of the EEC should specify this acceleration in the relative NAD regarding the Eurocode 8.

For example, in research project SA/419 calculation examples the following values have been assumed for Italy:

\[ \frac{a}{g} = 0.15 \quad weak \ earthquakes \]

\[ 0.25 \quad medium \ earthquakes \]

\[ 0.35 \quad strong \ earthquakes \]

* row 6

For structures that meet the "regularity" criteria, it is possible to perform a "simplified modal analysis" in which the seismic actions are modelled by a set of horizontal storey forces. Nevertheless, it is always possible to proceed directly with the more sophisticated "multi-modal analysis" that requires a computer code for dynamic analysis.

* row 8

- In the case of a multi-modal analysis the earthquake actions are applied indirectly by loading the structure with a given acceleration defined by the value of the design spectrum.

In this case we do not perform a direct determination of a set of design loads to apply to the frame.
2.4.10 - FC R2

* row 2

- The earthquake loads are considered as accidental actions and are thus combined with the other actions with different load combination rules than those adopted in non seismic design.

The ULS seismic combination is thus very similar to the one given in the EC1 for the "combinations for accidental design situations":

\[ \sum_j \gamma_{G_j} \cdot G_{k,j} + A_d + \Psi_{1i} \cdot Q_{k,i} + \sum_i \Psi_{2i} \cdot Q_{k,i} \]

where:

- \( A_d \): design value of the accidental action;
- \( G_{k,j} \): characteristic values of the permanent actions;
- \( Q_{k,i} \): characteristic value of the dominant variable action;
- \( Q_{k,i} \): characteristic values of the other variable actions;
- \( \Psi_{1i, \Psi_{2i}} \): combination coefficients;
- \( \gamma_{G_j} \): partial factors for the permanent actions.

* row 4

- In seismic design, the EC8 indicates a ULS load combination to be adopted for both ULS and SLS analysis.

Nevertheless, to take into account the lower return period of the seismic event associated with the SLS (with respect to the ULS earthquake), in the SLS checks appropriate correction factors are introduced (e.g. see the coefficient "v" in the SLS horizontal storey deflection checks in FC R4, row 9).
The class of cross sections of the profiles can be easily estimated: there are many profiles catalogues that indicate the cross section class for pure bending or for pure compression for different steel grades. It is then possible to select profiles whose ductility properties (i.e. cross section classes) are compatible with the value of the behaviour factor "q" assumed in the seismic analysis, as it is requested by the EC8.

2.4.12 - FC R4

*row 5*

- where:

\[ d_{\text{dis},i} \] : horizontal displacement of storey(i) due to the earthquake actions;
\[ d_{\text{res},i} \] : horizontal displacement of storey(i) due to the other horizontal and vertical actions.

*row 6*

- The check has to be performed for all the storeys of the frame.

*row 9*

EC8 - part 1.2

clause 4.2.2 • As it has been evidenced in FC R2, the "v" coefficient has been introduced to take into account the lower return period of an earthquake associated with the SLS (with respect to a ULS earthquake).

*row 11*

- The given limits refer to the case in which the non structural elements are fixed in such a way as not to interfere with the structural deformations.
• The check has to be performed for all the storeys of the frame.

2.4.13 - FC R8

• If the 2D-frame is considered as "non sway", the design rules for this type of structure are given in path 1 of FC 1 that corresponds to an ordinary first order analysis approach (the indications are given in ProfilARBED's "Design handbook according to Eurocode 3 for braced or non-sway steel buildings").

EC3 - 5.2.5.2(4) • The EC3 enables to adopt a simplified expression for the "sway-non sway" criterion. A 2D-frame results to be "non-sway" if all the storeys of the structure meet the following expression:

\[
\frac{1}{\alpha_{\sigma, j}} = \left( \frac{\delta_j}{h_j} \right) \left( \frac{\Sigma V_i}{\Sigma H_i} \right) \leq 0.1
\]

Vice versa, the frame is defined as "sway".

• The calculations examples performed during this work evidenced that the given formula gives incorrect results in the case of asymmetrical 2D-frames and/or asymmetrical vertical loads; in fact, for different load combinations with same vertical loads, the criterion gives different values for the storey "sway-non sway" ratios.

It is in fact certain that the sensibility of a structure to the second order effects depends only on the entity of the vertical loads; the horizontal loads have no interest from this point of view.

It is then evident that, being the above formula directly dependant on the horizontal loads (through \( \Sigma H \)), the criteria could result to give not very precise results.

This part of the EC3 should be re-studied thoroughly in order to meet the needs of cases such as asymmetrical frames.

* row 8
EC3 - 5.2.4.2(4) • The check for member imperfections needs to be performed only for the columns of rigid frames.

2.4.14 - R12.1

The "amplified sway moments method " analysis is given in path 2 of FC 1

* row 3

• The method is applicable if the given expression is fulfilled by all the storeys of the frame

* row 5

• The values of the amplification factors "α_i" (one for each storey) are directly related to the values of the "sway-non sway" criterion ratios.

In fact, "R_i" is given by:

\[ R_i = \left( \frac{\delta_i}{h_i} \right) \cdot \left( \frac{\Sigma V_i}{\Sigma H_i} \right) \]

* row 6

• where:

\[ M_i(H) \] : portion of the first order moments due to the horizontal loads (global frame imperfections included);

\[ M_i(V) \] : portion of the first order moments due to the vertical loads.
• The definition of "sway moments" is not very clear in the case of asymmetrical 2D-frames and/or asymmetrical vertical loading; in fact it is not possible to evaluate the part of the moments due to the vertical loads that is related to the horizontal "sway" of the frame and the remaining part related to the vertical deflections of the beams. For this reason, the method is not correctly applicable in this particular case.

• A correct formulation of the problem, instead, would enable to determine the values of the second order moments in beams and columns with the following formula:

\[ M_{Hi} = \alpha_i \cdot M_i(H) + \alpha_i \cdot M_i^{SW}(V) + M_i^{BD}(V) \]

where:

- \( M_i(H) \) : as given above;
- \( M_i^{SW}(V) \) : share of the first order moments due to the vertical loads related to the horizontal "sway" of the storeys;
- \( M_i^{BD}(V) \) : share of the first order moments due to the vertical loads related to the vertical deflections of the beams.

As specified above, the gap in the Eurocode 3 regards the determination of \( M_i^{SW}(V) \) and \( M_i^{BD}(V) \).

• Only the bending moments are incremented by the value of the amplification factors "\( \alpha_i \)"; For \( N \) and \( V \), instead, the values of the first order analysis are taken directly for the local member checks.

2.4.15 - FC R12.2

The "sway mode buckling length" analysis method is given in path 3 of FC 1

* row 6

• Similarly to the "amplified sway moments" method, the "sway mode buckling lengths" is not applicable for asymmetrical 2D-frames and/or asymmetrical vertical loading.
* row 7

- where:

\[ M_i(H) \] : share of the first order moments due to the horizontal loads (global frame imperfections included);
\[ M_i(V) \] : share of the first order moments due to the vertical loads.

* row 8

- Only the bending moments are incremented by the amplification factors "1.2"; For N and V, instead, the values of the first order analysis are taken directly for the local beam checks.

**EC3 - 5.2.6.2(8)**: the incremented value of the bending moments "\( M_{n,i} \)" should be adopted not only for beam design but also for the design of the elements of the beam-column connections.

2.4.16 - FC R12.3

The "second order" analysis method are given in paths 4 and 5 of FC 1

* row 2

**EC3 - Fig.5.5.1**: As specified in FC 1 and depending on the check for member imperfections (see FC R8, row 11), it may be necessary to take into account the member imperfections "\( e_{o.d} \)" when performing the second order analysis of the frame.

- For design purposes, it may be useful to replace the member displacement "\( e_{o.d} \)" with a set of equivalent loads that lead to the same deflected shape of the column:

\[
q = \frac{8 \cdot N_{sd} \cdot e_{o.d}}{L^2} \quad \text{DISTRIBUTED LOAD ALONG THE MEMBER}
\]

\[
Q = \frac{q \cdot L}{2} = \frac{4 \cdot N_{sd} \cdot e_{o.d}}{L} \quad \text{CONCENTRATED LOADS AT THE TWO ENDS}
\]
The "classification of cross sections" is a preliminary design activity to the local member checks that has to be performed with all the elements of the 2D-frame. It is based on the post-elastic and ductile characteristics of the profiles cross section and can performed from the geometrical properties of the cross section.

- The Eurocode 3 defines four different classes (1, 2, 3, and 4) with decreasing post-elastic characteristics.

A design simplification for common structures, to be considered only in the classification of cross sections, is to suppose for both beams and columns a simplified distribution of the internal forces and moments (i.e. for beams - pure bending, for columns - pure compression); in this way it is possible to avoid to determine the position of the neutral axis for each different combination of M and N.

This assumption results to be:

* For the columns - conservative since pure compression is more severe than mixed bending-axial forces;
* For the beams - being N very small, it is very close to the effective situation.

* row 9

- The values of M, N, V have been determined performing an elastic analysis (i.e. supposing that the stresses in the extreme fibres of the steel member are not greater than the yield stress); for this reason, only class 3 or 4 design methodologies have been adopted since only these two classes adopt elastic static values in the local member calculations.

- For class 4, the EC3 indicates that the resistance of the cross sections is influenced by local buckling phenomenon. For this reason only part of the compression elements of the cross section can be effectively considered to resist the internal forces.

Once these portions have been determined, it is possible to determine all the parameters of class 4 cross sections (e.g. $A_{\text{eff}}$, $W_{\text{eff}}$, etc).

* rows 10, 11, 12

- The local member checks to be performed have been divided in two different groups, the first regarding the beams, the second regarding the columns.

2.4.18 - FC R16.1

* row 3

- "Nsd" is determined from the global analysis of the 2D-frame adopting one of the methods of analysis given by the EC3 (see FC 1 - paths 2, 3, 4, 5 and 6).
2.4.19 - FC R16.2

* row 3

- "Msd" is determined from the global analysis of the 2D-frame adopting one of the methods of analysis given by the EC3 (see FC 1 - paths 2, 3, 4, 5 and 6).

* row 5

- where:
  
  $A_t$: tension flange area;
  $A_{t,\text{net}}$: tension flange net area;
  $A_t$: complete tension zone area (tension flange + tension zone of the web);
  $A_{t,\text{net}}$: complete tension zone net area (tension flange + tension zone of the web);

* rows 6, 7, 8, 9

- The two checks verify whether it is necessary to take into account the bolts holes or not in the determination of the section properties; the criteria is performed twice, once for the tension flange, the second time for the tension zone of the web.

  It is obvious that these calculations are required only if the tension flange and/or tension zone of the web have holes for bolts.

2.4.20 - FC R16.3

* row 3

- "Vsd" is determined from the global analysis of the 2D-frame adopting one of the methods of analysis given by the EC3 (see FC 1 - paths 2, 3, 4, 5 and 6).

* row 6

- It is obvious that this calculations is required only if the section has holes for bolts.
• The two formulas for the determination of \( V_{\text{pl.Rd}} \) refer both to class 3 and 4 cross sections.

2.4.21 - FC R16.4

* row 3

• \( N_{\text{sd}} \) and \( M_{\text{sd}} \) are determined from the global analysis of the 2D-frame adopting one of the methods of analysis given by the EC3 (see FC 1 - paths 2, 3, 4, 5 and 6).

2.4.22 - FC R16.5

* row 3

• \( M_{\text{sd}} \) and \( V_{\text{sd}} \) are determined from the global analysis of the 2D-frame adopting one of the methods of analysis given by the EC3 (see FC 1 - paths 2, 3, 4, 5 and 6).

• \( V_{\text{pl.Rd}} \) refers to the shear cross sectional resistance calculation of members (see FC R16.3 - row 8).

2.4.23 - FC R16.6

* row 3

• \( N_{\text{sd}} \), \( M_{\text{sd}} \) and \( V_{\text{sd}} \) are determined from the global analysis of the 2D-frame adopting one of the methods of analysis given by the EC3 (see FC 1 - paths 2, 3, 4, 5 and 6).

* row 4
• "V_{pl,Rd}" refer to the shear cross sectional resistance calculation of members (see FC R16.3 - rows 8).

2.4.24 - FC R16.7

* row 2

• "V_{sd}" is determined from the global analysis of the 2D-frame adopting one of the methods of analysis given by the EC3 (see FC 1 - paths 2, 3, 4, 5 and 6)

* row 3

• where:

  d : depth of the web;
  tw : web thickness.

* row 6

• where:

  a : clear spacing between web transverse stiffeners

* row 15

• If the check is not fulfilled it is possible to:
  a) re-design the stiffeners, normally assuming a smaller value for "a" or introducing stiffeners in non stiffened webs;
  b) changing profiles for the elements.

2.4.25 - FC R16.8

* row 2
- "Nsa" and "Msa" are determined from the global analysis of the 2D-frame adopting one of the methods of analysis given by the EC3 (see FC 1 - paths 2, 3, 4, 5 and 6).

* row 3

- where:

  A : cross sectional area;
  i : radius of gyration.
3 - SECTION B: CALCULATION EXAMPLES ON SWAY FRAMES

3.1 - GENERAL

As it has been specified above, some calculation examples on 2D steel frames have been performed. These are simply practical applications of the design procedures illustrated in Section A.

In order to link the two sections of the handbook, at the beginning of each phase of the calculation examples is given a copy of the general flow chart "FC 1" in which the relevant box of the current calculation phase has been shaded.

For each different structure two calculations have been performed: one in which the seismic actions have not been considered (and thus the wind loads are the only important horizontal actions), the other considering the loads due to the earthquakes and thus adopting different sections for the steel members.

3.2 - THE CALCULATIONS

In this handbook, only a few representative examples have been included:

- Civil building (CIV) - Non seismic calculation
- Civil building (CIV) - Seismic calculation
- Multi-storey parking building (PAR) - Non seismic calculation
- Multi-storey parking building (PAR) - Seismic calculation

These are given in Annex II.

As it has been evidenced before, some of the rules regarding "sway frame" design according to the EC3 are not correctly applicable in the case of asymmetrical frames. This fact has been evidenced in the calculation examples of the civil building. The calculations regarding the multi-storey parking, instead, are based on design clauses that are perfectly defined in the case of symmetrical structures.
4 - SECTION C: THE COMPUTER PROGRAMS

4.1 - GENERAL

As it has been evidenced above, the second order computer codes are necessary when a rigorous second order analysis of the structural model is performed. The popularity of these programs among the common engineering offices is strictly related to the hardware platforms that are available for the design activities. The second order programs, in fact, require powerful computers; in addition, the duration of the second order analysis is much longer than for first order calculations since it is based on an iterative process for the determination of the structural displacements.

Nevertheless, once these codes will become of common design practice it will be possible to perform directly second order analysis and obtain values for the internal forces and moments with a higher degree of precision.

4.2 - THE SECOND ORDER COMPUTER PROGRAMS

In Annex III are given the main characteristics of some of the major second order computer programs available in the European market. These can be of two different types:

- Multi-purpose computer codes;
- Specific codes for beam-column frames.

The common first order analysis programs have not been included in this list since these are already very well known by the designers of the steel structures.
ANNEX I

The Flow Charts
Flow-chart (FC 1): Elastic global analysis of steel frames according to Eurocode 3 (Details)

1. Determination of load arrangements (EC 1 and EC 8)
   - Lead cases for ULS [2.3.3]
   - Lead cases for SLS [2.3.4]

2. Predesign of members, beams & columns => Sections with pinned and/or rigid connections

3. ULS checks
   - Frame with bracing system [Chap. 5]
   - Classification of the frame
   - Braced frame
   - Global imperfections of the frame [5.2.5.3]
   - Non-sway frame
     - $\delta_m \leq 0.2 \delta_n$ [5.2.5.1 (7)]
     - $V_{ld} / V_{cr} \leq 0.1$ [5.2.6.2 (4)]
   - Sway frame
     - $L > 0.5 [A_{Sy} / N_{sd}]^{0.5}$ [5.2.4.2 (4)]

4. SLS checks
   - Frame without bracing system
   - Classification of the frame
   - Global imperfections of the frame [5.2.5.3]
   - Non-sway frame
     - $V_{ld} / V_{cr} \leq 0.1$ [5.2.6.2 (4)]

5. FIRST ORDER ANALYSIS
   - Non-sway mode buckling length approach [5.2.6.2 (1a)]
   - Sway mode buckling length approach [5.2.6.2 (1b)]
   - Non-sway mode with sway moments amplified by factor $1 / (1 - V_{sd} / V_{cr})$ [5.2.6.2 (3)]
   - Sway mode with sway moments amplified by factor 1.2 in beams & connections

6. SECOND ORDER ANALYSIS
   - Members imperfections end
   - Members with end
     - Checks of the in-plane stability: members buckling [Chap. 5.5]
     - Checks of the out-of-plane stability: members and/or frame buckling [Chap. 5.5]
     - Checks of resistance of cross-sections [Chap. 5.4]
     - Checks of local effects (buckling and resistance of webs) [Chap. 5.6 and 5.7]
     - Checks of connections [Chap. 6 and Annex f]
Flow chart: Determination of loads according to the EC1 and to the EC8

- **DETERMINATION OF LOADS (EC1 - EC8)**
  - Design of non-structural elements
  - **CALCULATION OF ACTIONS**
    - **PERMANENT ACTIONS**
      - Self weight loads (EC1 - part 2.1 - chap. 2)
      - Dead loads (EC1 - part 2.1 - chap. 2)
      - Frame imperfection loads (EC3 - 5.2.4.3)
      - Imposed loads (EC1 - part 2.1 - chap. 4)
    - **VARIABLE ACTIONS**
      - Snow loads (EC1 - part 2.1 - chap. 5)
      - Wind loads (EC1 - part 2.1 - chap. 6)
    - **ACCIDENTAL ACTIONS**
      - Earthquake loads (EC8 - part 1.2 - chap. 3)
DETERMINATION OF SELF WEIGHT LOADS (EC1 - part 2 / chap. 2)

Determine profiles self weight

Account for joint elements self weight

Increment of profiles self weight

DESIGN SELF WEIGHT LOADS (SWL)
DETERMINATION OF DEAD LOADS
(EC1 - part 2.1 - chap.2)

1. Determine each single dead load per unit
   (slabs, secondary steel members, cladding, ceiling covering, pavements, etc)

2. Determine the relevant portion of building related
to the 2D-frame

3. Compute 2D-frame storey loads

4. DESIGN DEAD LOADS (DL)
DETERMINATION OF THE EQUIVALENT FRAME IMPERFECTION LOADS

1. Compute frame total vertical storey loads $W_i$
2. Compute $k_e, k_s$ ($EC3 - 5.2.4.3(1))$
3. Compute initial sway imperfection $\phi$ ($EC3 - 5.2.4.3(1))$
4. Compute equivalent storey horizontal forces $F_i = \phi * W_i$ ($EC3 - 5.2.4.3(7))$
5. Design equivalent frame imperfection loads (FIL)
DETERMINATION OF IMPOSED LOADS (EC1 - part 2.1 - chap. 4)

1. Determine the imposed loads building category (sections 4.5 - 4.6)
2. Determine the characteristic load value $q_k$ (sections 4.5 - 4.6)
3. Determine the building width with reference to each 2D-frame $b$
4. Compute the storey imposed loads $q = q_k b$
5. Design imposed loads (IL)
DETERMINATION OF SNOW LOADS
(EN1 part 2.1 - chap.5)

Determine the snow load shape coefficient $\mu$
(section 5.7)

Determine exposure and thermal coefficients $C_e$, $C_t$
(clauses 5.5.2 - 5.5.3)

Determine value of the snow load on ground $s_a$
(section 5.4; annex A)

Compute the snow load on roof $s$
(clause 5.5.1)

- yes $s \leq q_k$
- no

THE ROOF STOREY SHOULD BE LOADED WITH THE IMPOSED LOADS INSTEAD

Determine the building width with reference to each 2D-frame $b$

Compute the storey snow loads:
$s' = s \cdot b$  ($s' = q_k \cdot b$)

DEFINE DESIGN SNOW LOADS (SL)
Flow chart FC R1.6: Determination of wind loads (WL)

Determination of wind loads
(EN 1 - part 1 - chap. 6)

1. Compute diagonal size of loaded area \( l \)
2. Classify terrain roughness (annex A.4)
3. Compute exposure coefficient \( C_e(z) \) (clause 6.7.5)
4. Compute reference wind speed \( V_{\text{ref}} \) (annex A.9)
5. Compute reference wind pressure \( q_{\text{ref}} \) (clause 6.7.1)
6. Compute design wind pressure \( w_e(z) \) (clause 6.7.3)
7. Compute pressure coefficients \( C_p \) (annex B.1)
8. Compute size coefficient \( C_{\text{size}} \) (clause 6.7.6)
9. Compute reference wind pressure \( q_{\text{ref}} \) (clause 6.7.1)
10. Determine the lateral area of building related to the frame storey \( A_i \)
11. Compute 2D-frame storey loads \( F_i = w_e(z_i) \cdot A_i \)
12. Design wind loads (WL)
DETERMINATION OF EARTHQUAKE ACTIONS

1. Classify subsoil class
2. Classify steel frame
3. Determine structural viscous damping ratio $\xi$
4. Compute frame natural period $T_o$
5. Determine ground acceleration $a/g$
6. Determine damping correction factor $\eta$
7. Compute design value of spectrum $\beta(T_0)$
8. Check for simplified modal analysis
   - $T_o \leq 2T_1$
9. DESIGN EARTHQUAKE ACTIONS (EL)

SIMPLIFIED MODAL ANALYSIS

1. Determine load combination coefficient $\psi_1$
2. Determine storey load factors $Q_j$
3. Compute design storey loads $W = W_{Fj} + W_{Ej} + \psi_1 \times Q_j$
4. Compute storey horizontal forces $F_i$

MULTI-MODAL ANALYSIS

1. Perform dynamic analysis with design spectrum $\beta(T)$
COMBINATION OF ACTIONS

Type of design

- non seismic -

ULS

\[ \sum \gamma G_{ij} + 1.35 \sum Q_{ki} \] (EC1 - part 1 - 9.4.3)

SLS

\[ \sum G_{ij} + 0.9 \sum Q_{ki} \] (EC1 - part 1 - 9.3.5)

- seismic -

ULS

SLS

\[ \gamma E + \sum (G_{ij} + \sum P_{ij} + Q_{ki}) \] (EC8 - part 1.1 - 6.2)

DESIGN LOAD COMBINATIONS
Flow chart FC R3  Pre-design of members

PREDESIGN OF MEMBERS

Preliminary calculations for the determination of member sections

Assume profiles for member sections

Type of analysis

- seismic
- non seismic

Is the estimated class of cross-sections compatible with the assumed value of the behaviour factor \( \mu \)?

(\textit{EC8 - part 1.3 - 3.5.2.1})

ADOPT THE ASSUMED PROFILES
Flow chart

FC R4

Serviceability limit states (SLS) checks

FC R3
row 3

The predesign of members has to be performed again

SLS CHECKS
(EC3 - chap. 4)

Type of analysis

non seismic

seismic

Perform an ordinary first order analysis combining the actions with the SLS equation
(EC1 - part 1 - 9.5.5)

single storey

Type of building

Perform an ordinary first order analysis combining the actions with the ULS equation
(EC8 - part 1.1 - 6.2)

Compute dsi,j , dres,i

Assume v = 2.5

no

yes

di ≤ h/300 (EC3 - 4.2.2(4))

ho ≤ h/300 (EC3 - 4.2.2(4))

dot ≤ h/500 (EC3 - 4.2.2(4))

yes

no

yes

no

SLS CHECKS FULFILLED
Flow chart: Sway-non sway criterion and check for member imperfections

Sway - Non Sway Criterion: Check for Member Imperfections

EC3 - sections 5.2.5.2 and 5.2.4.2)

Type of analysis

- Non seismic
- Seismic

Perform an ordinary first order analysis combining the actions with the ULS equation
(EC3 - part I - 9.4.3)

Perform an ordinary first order analysis combining the actions with the ULS equation
(EC3 - part I - 6.2)

Compute $d_{li}$, $d_{ri}$

Compute $d_{li}$, $d_{ri} = d_{li} - d_{ri}$

Sway - Non Sway Criterion
(EC3 - 5.2.5.2(4) - EC8 - part 1.2 - 4.1.1.2)

$\frac{L_i}{h_i} \left( \frac{E_f}{E_i} \right) \leq 0.1$

Member subject to axial compression with moment resisting connections?

Check for Member Imperfections
(EC3 - 5.2.4.2(4))

Compute non dimensional slenderness

$\tilde{\lambda} = \frac{\lambda}{(EC3 - 5.5.1.2(1))}$

$\lambda \leq 0.5 \cdot \sqrt{\frac{A_f}{N_e}}$

(EC3 - 5.2.4.2(4))

A Second Order Global Analysis with account for the Member Imperfections has to be performed

A First/Second Order Global Analysis with no Account for the Member Imperfections can be performed
Perform an ordinary first order analysis combining the actions with the ULS equations

\[ 0.1 < (\delta / h_i)(\Sigma V_i / \Sigma H_i) \leq 0.25 \]

(Amplified sway moments method)
SWAY MODE BUCKLING LENGTHS METHOD
(EC3 - section 5.2.6.2(8))

Perform an ordinary first order analysis combining the actions with the ULS equations
(EC1 - part 1 - 9.4.5 / EC8 - part 1.1 - 6.2)

Type of member

Non symmetrical frame and/or non symmetrical vertical loading

Symmetrical frame with symmetrical vertical loading

Type of 2D frame and loading

THE EC3 DOES NOT GIVE CLEAR INDICATIONS FOR THIS CASE

Adopt first order analysis values for M, N, V

Compute $M_{i(H)}$, $M_{i(V)}$

Compute

$M_{li} = 1.2\cdot M_{i(H)} + M_{i(V)}$
(EC3 - section 5.2.6.2(8))

Adopt the in plane buckling lengths for the sway mode in column design
(EC3 - section 5.2.6.2(8))

Adopt these as design values of the bending moments for beam design
SECOND ORDER ANALYSIS METHOD
(EC3 - section 5.2.6.2(2))

Perform a second order analysis combining the actions with the ULS equations. Account for member imperfections in the columns where this is necessary (according to FC R8, row 11).
(EC1 - part 1 - 9.4.5. & EC8 - part 1.1 - 6.2)

Adopt the second order analysis values of M, N, V for member design

Adopt the in plane buckling lengths for the non sway mode for column design
(EC3 - section 5.2.6.2(2))
CROSS SECTIONAL AND BUCKLING RESISTANCE OF MEMBERS
(EC3 - sections 5.4, 5.5, 5.6)

CLASSIFICATION OF CROSS SECTIONS
(EC3 - section 5.3)

Type of member

Assume beam subject to pure bending

Compute \( f \)
(table 5.3.1 - sheet 1)

Determine web class \( C_w \)
(table 5.3.1 - sheet 3)

Determine flange class \( C_f \)

Assume column subject to pure compression

\( C = \text{MAX} (C_w, C_f) \)

Assume \( C = 3 \)

\( C = 1, 2 \)

\( C = 4 \)

Compute effective cross section properties
(section 5.3.5)

Type of member

Assume beam subject to pure bending

Assume column subject to pure compression

CROSS SECTIONAL AND BUCKLING CHECKS FOR BEAMS

M RESISTANCE (section 5.4.5)
V RESISTANCE (section 5.4.6)
M+V RESISTANCE (section 5.4.7)
V BUCKLING (section 5.6)

CROSS SECTIONAL AND BUCKLING CHECKS FOR COLUMNS

N RESISTANCE (section 5.4.4)
M RESISTANCE (section 5.4.5)
V RESISTANCE (section 5.4.6)
M+N RESISTANCE (section 5.4.8)
M+V RESISTANCE (section 5.4.9)

M+N BUCKLING (section 5.5.4)
Compression cross sectional resistance of members

The predesign of the member has to be performed again.

- Determine $N_{sd}$
  - Determine $f_y$
  - $C = 3$
  - Member cross section class
  - Determine $A$
    - Compute $N_{c,Rd} = A f_y / \gamma_M$ (5.4.4(2))
  -  
  - $N_{sd} \leq N_{c,Rd}$

- Determine $A_{eff}$
  - Compute $N_{c,Rd} = A_{eff} f_y / \gamma_M$ (5.4.4(2))
  - $C = 4$

CHECK FULFILLED
The predesign of the member has to be performed again.

**BENDING CROSS SECTIONAL RESISTANCE OF MEMBERS**

(IEC3 - section 5.4.5)

1. Determine $M_{sd}$
2. Determine $f_y$, $f_u$
3. Compute $A_t$, $A_{mat}$, $A_i$, $A_{mat}$

**For the tension flange**

\[
\frac{(f_y/f_u)[V_{M2}/T_{M0}]}{0.9[A_{mat}/A_i]} \leq 0.9\frac{[\Delta]}{A_{mat}}
\]

(IEC3 - clause 5.4.5.11)

- **THE BOLTS HOLES SHALL NOT BE ALLOWED FOR IN THE TENSION FLANGE WHEN CALCULATING THE SECTION PROPERTIES**

**For the tension zone of the web**

\[
\frac{(f_y/f_u)[V_{M2}/T_{M0}]}{0.9[A_{mat}/A_i]} \leq 0.9\frac{[\Delta]}{A_{mat}}
\]

(IEC3 - clause 5.4.5.13)

- **THE BOLTS HOLES SHALL NOT BE ALLOWED FOR IN THE TENSION ZONE OF THE WEB WHEN CALCULATING THE SECTION PROPERTIES**

\[
C = 3 \quad \text{Member cross section class}
\]

- **Determine $W_{t}$**
- **Determine $W_{t}$**
- **Determine $W_{mat}$**
- **Determine $W_{mat}$**

\[
M_{Rd} = W_{t} f_y / \gamma_{M0}
\]

(IEC3 - 5.4.5.2(1))

- **Compute**
- **Compute**
- **Compute**
- **Compute**

\[
M_{Rd} = W_{t} f_y / \gamma_{M0}
\]

(IEC3 - 5.4.5.2(1))

\[
M_{Rd} = W_{mat} f_{y} / \gamma_{M0}
\]

(IEC3 - 5.4.5.2(1))

\[
M_{Rd} = W_{mat} f_{y} / \gamma_{M0}
\]

(IEC3 - 5.4.5.2(1))

\[
M_{Rd} = W_{mat} f_{y} / \gamma_{M0}
\]

(IEC3 - 5.4.5.2(1))

\[
M_{Rd} = W_{mat} f_{y} / \gamma_{M0}
\]

(IEC3 - 5.4.5.2(1))

\[
M_{sd} \leq M_{ad}
\]

\[
\text{CHECK FULFILLED}
\]
Flow chart FC R16.3

Shear cross sectional resistance of members

SHEAR CROSS SECTIONAL RESISTANCE OF MEMBERS (EC3 - section 5.4.6)

1. Determine $V_{sd}$
2. Determine $f_y$, $f_u$
3. Compute $A_v$, $A_{v, net}$
4. Check $(fy/fu) \times A_v \leq A_{v, net}$ (EC3 - clause 5.4.6(8))
5. The bolts holes shall not be allowed for when calculating the section properties
6. Compute $V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma M_0$ (EC3 - 5.4.6(1))
7. The bolts holes shall be allowed for when calculating the section properties
8. Compute $V_{pl,Rd} = A_{v, net} \times (f_u / \sqrt{3}) / \gamma M_0$ (EC3 - 5.4.6(8))
9. $V_{sd} \leq V_{pl,Rd}$
10. Check fulfilled
The predesign of the member has to be performed again.

**Flow chart Bending-axial cross sectional resistance of members**

1. **Flow chart FC R3**
   - The predesign of the member has to be performed again.

2. **Flow chart FC R16.4**
   - BENDING-AXIAL CROSS SECTIONAL RESISTANCE OF MEMBERS
   - (EC3 - section 5.4.4)
   - Determine $N_{sd}$, $M_{sd}$
   - Determine $f_y$
   - Member cross section class $C = 3$
   - Determine $A$, $W_{el}$
   - Check
     \[
     \left( \frac{N_{sd}}{A + f_y \gamma_{M0}} \right) + \left( \frac{M_{sd}}{W_{el} f_y \gamma_{M0}} \right) \leq 1
     \]
     (EC3 - 5.4.8.2(2))
   - Check
     \[
     \left( \frac{N_{sd}}{A_{eff} f_y \gamma_{M11}} \right) + \left( \frac{M_{sd}}{W_{eff} f_y \gamma_{M11}} \right) \leq 1
     \]
     (EC3 - 5.4.8.3(2))
   - CHECK FULFILLED

3. **Flow chart FC R16.4**
   - BENDING-AXIAL CROSS SECTIONAL RESISTANCE OF MEMBERS
   - (EC3 - section 5.4.4)
   - Determine $N_{sd}$, $M_{sd}$
   - Determine $f_y$
   - Member cross section class $C = 4$
   - Determine $A_{eff}$, $W_{eff}$
   - Check
     \[
     \left( \frac{N_{sd}}{A_{eff} f_y \gamma_{M11}} \right) + \left( \frac{M_{sd}}{W_{eff} f_y \gamma_{M11}} \right) \leq 1
     \]
     (EC3 - 5.4.8.3(2))
   - CHECK FULFILLED
Flow chart

**FC R16.5**

**Bending-shear cross sectional resistance of members**

(EC3 - section 5.4.7)

1. **BENDING AND SHEAR CROSS SECTIONAL RESISTANCE OF MEMBERS**

   - Determine $V_{sd}, M_{sd}$
   - Recall $V_{pL,Rd}$

2. **yes**

   - $V_{sd} \leq 0.5 \cdot V_{pL,Rd}$
     (EC3 - clause 5.4.7(3))

3. **no**

   - **IT IS NECESSARY TO CHECK THE INFLUENCE OF THE SHEAR FORCE ON THE BENDING RESISTANCE**

4. Compute $p$
   (EC3 - 5.4.7(3)(a))

5. **check fulfilled**

6. **Member cross section class**

7. **C=3**

8. Compute $W_{ei}, A_v$

9. $M_{V,Rd} = \left[ W_{ei} - \left( p \cdot A_v E_0 \right) \right] \left( f_y / \gamma_M \right)$
   (EC3 - 5.4.7(3)(a))

10. **no**

11. **yes**

   - $M_{sd} \leq M_{V,Rd}$

   - **CHECK FULFILLED**
The predesign of the member has to be performed again

BENDING, SHEAR AND AXIAL CROSS SECTIONAL RESISTANCE OF MEMBERS

Determine $N_{sd}$, $M_{xl}$, $V_{id}$

Recall $V_{pt}$, $R_0$

Compute $A$, $W_{at}$, $A^*$

Check

$N_{sd} / [A^{*} - A_f] + (M_{sd} / [W_{at} - (p^2 A + 2A + 4p + 2p^2)]) \leq 1$

Computes $p$

Compute $A_{err}$, $W_{at}$, $A^*$

$N_{sd} / [A^{*} - A_f] + (M_{sd} / [W_{at} - (p^2 A + 2A + 4p + 2p^2)]) \leq 1$

Computes $A^*$

Member cross section class

IT IS NOT NECESSARY TO CHECK THE INFLUENCE OF THE SHEAR FORCE ON THE BENDING-AXIAL RESISTANCE

IT IS NECESSARY TO CHECK THE INFLUENCE OF THE SHEAR FORCE ON THE BENDING-AXIAL RESISTANCE

Flow chart

FC R16.6

Bending, shear and axial cross sectional resistance of members
Flow chart SHEAR BUCKLING RESISTANCE OF MEMBERS (EC3 - section 5.6)

1. Determine \( V_{sd} \)
2. Compute \( d/w \)
3. Compute \( \varepsilon \) (5.6.1(7))

Type of web stiffening

- Web without stiffeners
  - Compute \( K_t \) (5.6.3(3))
  - Determine \( \alpha \)
  - Compute \( K_t \) (5.6.3(3))
  - Compute \( d/w \leq 69\varepsilon \) (5.6.1(1))
  - Compute \( d/w \leq 30\varepsilon \) + \( \sqrt{K_t} \) (5.6.1(1))

- Web with stiffeners
  - IT IS NOT NECESSARY TO PERFORM THE SHEAR BUCKLING CHECK FOR THE WEB

Type of web stiffening

- Web with stiffeners
  - SIMPLE POST-CRITICAL METHOD (EC3 - section 5.6.3)
  - Compute \( \lambda \) (5.6.3(2))
  - Compute \( V_{sd} - d/w \times t / \gamma_M \) (5.6.3(1))

IT IS NECESSARY TO RE-DESIGN THE STIFFENERS OR TO CHANGE SECTION FOR THE MEMBER

CHECK FULFILLED

THE WEB HAS TO BE PROVIDED WITH TRANSVERSE STIFFENERS AT THE SUPPORTS (5.6.1(1))

IT IS NECESSARY TO RE-DESIGN THE STIFFENERS OR TO CHANGE SECTION FOR THE MEMBER
The predesign of the member has to be performed again.

Determine N\text{td}, M\text{ld}

Determine A, t

\( f_y \)

Determine \( \beta_M \)

(figure 53.3)

Determine the relevant buckling curve for the column cross-section

(figure 53.3)

Determine \( \alpha \)

(figure 53.1)

Compute column buckling length 1

(annex E - section E.2)

Compute column stiffness

(\( \lambda = 1/1 \))

Compute \( \lambda, \phi, \kappa, \mu, K \)

(section 5.5.4)

C = 3

Member cross section class

Determine \( W_{\text{ct}} \)

Check

\[
\frac{N_{\text{sd}}}{K \cdot A_{\text{er}} f_y} + \left( K \cdot M_{\text{sd}} / (W_{\text{eff}} f_y) \right) \leq 1
\]

(Ec3 - 53.4(3))

Yes

C = 4

Determine \( A_{\text{fr}}, W_{\text{eff}} \)

Check

\[
\frac{N_{\text{sd}}}{K \cdot A_{\text{fr}} f_y} + \left( K \cdot M_{\text{sd}} / (W_{\text{eff}} f_y) \right) \leq 1
\]

(Ec3 - 53.4(5))

Yes

CHECK FULFILLED

No

The predesign of the member has to be performed again.
ANNEX II

The Calculation Examples
CIVIL BUILDING (CIV)

NON SEISMIC CALCULATION
Type of building: Civil (CIV)
Type of design: Non seismic

BUILDING AND STRUCTURAL DATA

- **Dimensions**
  - Length (m): 40
  - Width (m): 11.5
  - Height (m): 7.0

- **Storeys**
  - Number: 2

- **Structural system**
  - Transversal direction: the bracing system for the horizontal loads is given by nine beam-column rigid jointed frames whose distance is 5 m. The columns of these structures are supposed to be hinged at the bases.
  - Longitudinal direction: the bracing system is obtained with a number of cross bracing structures that are added to the frame that is supposed to have hinged joints.

MATERIAL

- Steel Grade (for profiles, plates and welds): Fe 510
- Bolts grade: 8.8
I - GENERAL INFORMATION

The structural model of the frame is given in fig.1. The geometry of the structure and the position of the columns was decided taking into account the rooms layout of these residential buildings. In fig.2 is shown the typical arrangement of the single units.

The described calculations refer to one of the nine transversal frames of the building; in this way we refer to a simplified two dimensional model of the structure. The joints between the beams and the columns were supposed to be full strength connections and the loads that have been considered refer to a portion of building whose width is equal to the distance between the frames (thus, in this case, we take into account a portion of 5 m).

To facilitate the comprehension of the calculations, at the beginning of each section is reproduced a copy of the general flow chart "FC1" in which is evidenced the design phase that is currently being carried out.

II - LOADS

The following loads have been considered:

A) Self weight loads (SWL)

These include the weight of the steel frame members only and are calculated automatically by the codes used in this calculation example ("Stranger" and "PEP micro").

To take into account the weight of the joint elements (i.e. end plates, stiffeners, welds, bolts) these elementary self weight loads have been increased by 10% in the analysis and thus a $\lambda = 1.1$ load factor is always adopted.
**B) Dead loads (DL)**

*Levels I-II*

- r.c. slab \(3 \text{ KN/m}^2\)
- secondary steel members self weight \(0.5 \text{ KN/m}^2\)
- partition walls and ceiling covering \(0.5 \text{ KN/m}^2\)
- pavement elements \(0.5 \text{ KN/m}^2\)

Total uniform loads \(4.5 \text{ KN/m}^2\)

Total uniform load for a 5 m wide building portion

\[ 5 \times 4.5 = 22.5 \text{ KN/m} \]

**C) Imposed loads (IL)**

*Levels I (and also II if these actions are greater than the snow loads)*

clause 4.5.2 category A \(2 \text{ KN/m}^2\)

Total uniform load for 5 m building portion

\[ 2 \times 5 = 10 \text{ KN/m} \]

**D) Snow loads (SL)**

*Level II*

The snow loads are given by:

\[ S = \mu_i \cdot C_e \cdot C_t \cdot S_k \]

In our case:

clause 5.7.2 \(\mu_i = 0.8\) (monopitch roof with \(\alpha \leq 0\))

\(C_e = 1\)

\(C_t = 1\)

annex A - A10 \(S_k = 0.9 \text{ KN/m}^2\) (Genova, Italy, \(A = 0\))

\[ S = 0.8 \cdot 1 \cdot 0.9 = 0.72 \text{ KN/m}^2 \]
It is important to add that in this case the imposed loads have been considered for the II storey, since these have a higher value than the snow loads.

**E) Wind loads (WL)**

The pressure acting on the external surfaces is:

\[ w_x(z) = c_p \cdot c_{size} \cdot c_x(z) \cdot q_{ref} \]

in which:

\[ c_{size} = 0.49 \left[ 1 + 0.063 \cdot \ln \left( \frac{3300}{l} \right) \right]^2 \]

\[ c_x(z) = c_r(z)^2 \cdot c_t(z)^2 \left[ 1 + \frac{1.4}{c_r(z) \cdot c_t(z)} \right] \]

in which:

\[ c_r(z) = K_r \cdot \ln \left( \frac{z}{z_0} \right) \quad \text{for } z > Z_{min} \]

\[ c_r(z) = c_r(z_{min}) \quad \text{for } 0 < z \leq Z_{min} \]

\[ c_t = 1 \quad (\text{for flat terrains}) \]

\[ q_{ref} = \frac{\rho}{2} \cdot v_{ref}^2 \]

in which:

\[ v_{ref} = v_{ref,0} \cdot (1 + K_a \cdot a_1) \cdot c_{tem} \cdot c_{dir} \]

\[ c_p \]

\[ \text{PRESSURE COEFFICIENT} \]

In our case, the input parameters are:

annex B - 1.2  
\[ c_p = +0.8 \quad \text{Upwind face} \]
\[ c_p = -0.3 \quad \text{Downwind face} \]

\[ l = \text{diagonal size of loaded area} \]
\[ l = \sqrt{7^2 + 40^2} = 40.6 \text{ m} \]

annex A - 4  
\[ K_r = 0.24 \]
\[ z_0 = 1 \text{ m} \]
\[ Z_{min} = 16 \text{ m} \]
\[ \rho = 1.25 \text{ Kg/m}^3 \]
\[ V_{\text{ref},0} = 28 \text{ m/s} \quad \text{(for Genova, Italy, cat. 3)} \]
\[ K_a = 0.55 \]
\[ a_0 = 0 \]
\[ C_{\text{tem}} = 1 \]
\[ C_{\text{dir}} = 1 \]

The wind pressure has been represented by a set of storey loads. These forces, have been calculated with the Lotus 1-2-3 spreadsheet given in table 1, that performs a simple execution of the above written formulas. These actions were subsequently applied to each level taking into account that +0.8 corresponds to the upwind face and -0.3 corresponds to the downwind face of the buildings.

**EC3 - 5.2.4.3**  
**F) Frame imperfections loads (FIL)**

**Levels I-II**

The initial frame imperfections may be represented by a set of horizontal forces that are given, for the generic storey \( i \), by:

\[ F_i = \phi \cdot W_i \]

where:

\[
\phi = \frac{k_s \cdot k_c}{200}
\]

\[ W_i \quad \text{TOTAL VERTICAL STOREY LOADS} \]

\[ k_c = \sqrt{0.5 + \frac{1}{n_c}} \quad \text{with } k_c \leq 1 \]

\[ k_s = \sqrt{0.2 + \frac{1}{n_s}} \quad \text{with } k_s \leq 1 \]

in which:

\( n_c = \text{number of columns} \)

\( n_s = \text{number of storeys} \)

In our case we have:

\( n_c = 3 \quad k_c = 0.913 \)

\( n_s = 2 \quad k_s = 0.837 \)
\[ \varphi = \frac{0.913 \cdot 0.837}{200} = 0.00382 \]

In the calculation of the total storey loads, it is necessary to sum to the vertical loads calculated in B) and C) the weight of the frame beams and columns; in our case we have supposed 0.5 KN/m2.

For both storeys we obtain:

\[ W = (4.5 + 2 + 0.5) \cdot 11.5 \cdot 5 = 402.5 \text{ KN} \]

The storey equivalent imperfection loads can now be calculated:

\[ F = 0.00382 \cdot 402.5 = 1.54 \text{ KN} \]

These horizontal loads are applied half in one side of the frame, the other half on the other.

**III - LOAD CASES**

The following load combination rules have been used:

**Clause 9.4.5**  
*A) Ultimate limit states (ULS)*

\[ \sum_{j} \gamma_{G_j} \cdot G_{k,j} + 1.35 \cdot \sum_{i} Q_{k,i} \]
B) Serviceability limit states (SLS)

\[ \sum G_{k,j} + 0.9 \cdot \sum Q_{k,i} \]

where:

- \( G_{k,j} \) = characteristic value of the permanent actions
- \( Q_{k,i} \) = characteristic value of the variable actions
- \( \gamma_{Gj} \) = partial factors for permanent actions

The loads considered in this calculation example are:

1) Permanent actions
   - SWL
   - DL
   - FIL

2) Variable actions
   - IL
   - SL
   - WL

It is important to notice that the equivalent frame imperfections loads have been considered as "permanent" actions since these characterise the geometrical imperfections of the frame elements and thus are a permanent characteristic of the structure.

In addition, since the frame geometry is asymmetrical, it was necessary to consider in the combinations the horizontal loads (WL and FIL) acting in both directions. The direction of these actions is indicated in every single case by adding "L" or "R" to the load case name.

It is also important to remember that the self weight loads (SWL) have already been increased by 10%.

The ULS load cases that have been considered in this calculation example are:

1) \( 1.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + 1.35 \cdot FIL \_ R \)
2) \(0.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + 1.35 \cdot FIL_R + 1.35 \cdot WL_R\)

3) \(0.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + 1.35 \cdot FIL_L\)

4) \(0.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + 1.35 \cdot FIL_L + 1.35 \cdot WL_L\)

The **SLS load cases** that have been considered are:

5) \(0.00 \cdot SWL + 1.00 \cdot DL + 0.9 \cdot IL + 1.00 \cdot FIL_R + 0.9 \cdot WL_R\)

6) \(0.00 \cdot SWL + 1.00 \cdot DL + 0.9 \cdot IL + 1.00 \cdot FIL_L + 0.9 \cdot WL_L\)

**IV - PRELIMINARY SIZING OF MEMBERS**

The pre dimensioning of the frame beams and columns was performed with simple preliminary calculations. The chosen profiles are shown in fig. 3.

EC3 - Chap. 4  **V - SLS CHECKS**
A preliminary first order analysis of the frame has been carried out. The numbering of nodes and members are given in fig.5. The SLS verifications that have been performed regard horizontal displacement checks of the different storeys of the frame.

According to the EC3, the horizontal deflection limits for multi-storey buildings are:

clause 4.2.2 (4) - **in each storey:**

\[ d_i \leq \frac{h_i}{300} \]

- **for the structure as a whole:**

\[ d_{tot} \leq \frac{h}{500} \]

where:

- \( d_{tot} \) = total frame horizontal deflection
- \( d_i \) = storey(i) relative horizontal deflection
- \( h \) = frame total height
- \( h_i \) = storey(i) height

The storeys horizontal deflections were obtained calculating the average value between the horizontal displacements of the two end nodes of each storey. It is also necessary to add that the verification of the storey relative horizontal displacement has only been performed in the first storey since it is less stiff than the second level and thus enables greater displacements.
In our case:

_for load case 5):

- for storey 1:
  \[ d_i = 0.895 \leq \frac{h_i}{300} = \frac{380}{300} = 1.27 \text{cm} \]

- for the structure as a whole:
  \[ d_{\text{tot}} = 0.971 \leq \frac{h}{500} = \frac{700}{500} = 1.40 \text{cm} \]

_for load case 6):

- for storey 1:
  \[ d_i = 1.10 \leq \frac{h_i}{300} = \frac{380}{300} = 1.27 \text{cm} \]

- for the structure as a whole:
  \[ d_{\text{tot}} = 1.39 \leq \frac{h}{500} = \frac{700}{500} = 1.40 \text{cm} \]

EC3
section 5.2

section 5.2.3.2 **A) CHECK FOR "SWAY -NON SWAY" BEHAVIOUR**
Before the internal forces and moments are calculated and used in member and cross-sectional checks, it is necessary to check if the non linear geometrical effects (second order) have to be taken into account or not.

A frame is classified as "sway" for a given load condition if, for at least one of the storeys, the following condition is satisfied:

\[
\frac{\delta_i}{h_i} \left( \frac{\sum V_i}{\sum H_i} \right) > 0.1
\]

where:
- \(h_i\) = storey(i) height
- \(\delta_i\) = relative horizontal displacement of storey(i) obtained from an ordinary first order analysis
- \(\sum V_i\) = total vertical reaction at the bottom of storey(i) from first order analysis
- \(\sum H_i\) = total horizontal reaction at the bottom of storey(i) from first order analysis

The values of the "sway-non sway" ratio for the ULS load cases (1, 2, 3, and 4) of this specific example are given in the Lotus 1-2-3- spreadsheets given in tables 2 and 3.

The frame results to be "sway" for load cases 2, 3, and 4; in these load cases it is necessary to take into account the non linear geometrical effects performing a second order analysis. For load case 1, instead, it is sufficient to take directly the first order internal forces and moments.

clause 5.2.4.2(4) **B) CHECK FOR MEMBER IMPERFECTIONS**
According to the EC3, for load cases in which the frame results to be "sway", it is necessary to check if the effects of member imperfections can be neglected or not.

For this purpose, if in a compression member with moment resisting connections the given formula is not fulfilled then it is necessary to take into account these imperfections in the global analysis.

\[
\bar{\lambda} \leq 0.5 \cdot \sqrt{\frac{A \cdot f_y}{N_{sd}}}
\]

where:

- \(\bar{\lambda}\) = in plane non dimensional slenderness calculated using a buckling length equal to the system length
- \(A\) = column cross section area
- \(N_{sd}\) = design value of the compression force
- \(f_y\) = design column yield strength

In the scope of this research project, the above written condition should be checked for all the columns of the frame, taking the internal forces and moments of load cases 2, 3 and 4.

As an example, we can take element 3 (HEA 280) with the internal forces of case 2.

In our case:
- \(A = 97.3\) cm\(^2\)
- \(N_{sd} = 586\) KN
- \(f_y = 355\) N/mm\(^2\)

\[
\bar{\lambda} = \frac{\lambda}{93.9 \cdot \varepsilon \cdot \sqrt{\beta_1}} = \frac{380}{93.9 \cdot 0.814} \cdot \sqrt{1} = 0.418
\]

And thus:

\[
\bar{\lambda} = 0.418 < 0.5 \cdot \sqrt{\frac{97.3 \cdot 355 \cdot 10^{-1}}{586}} = 1.214
\]

If the above written formula is fulfilled by the other columns of the frame, then it is not necessary to account for the member imperfections in the global analysis.
section 5.2.6.2  C) SECOND ORDER ANALYSIS

The EC3 enables to adopt the rigorous second order calculation method together with other indirect second order analysis methods.

5.2.6.2 (2)  1) Rigorous second order analysis method

A direct second order analysis has been with "PEP micro". The results of this calculation (internal forces and moments) are used directly for the local member and cross sectional checks that are performed using the "non sway" buckling lengths.

5.2.6.2 (8)  2) Sway mode buckling length method
This is one of the indirect second order analysis methods indicated by the EC3; for *column* member and cross-sectional checks we take directly the first order analysis results and adopt "sway mode" buckling lengths; for the *beams* it is necessary to increase by 20% the "sway moments"; the beams local member and cross-sectional checks are then performed with the increased values of these moments.

5.2.6.2 (3)  
3) Amplified sway moments method

This is an indirect second order calculation method that is applicable only if, for a given load case, all the storeys of the frame satisfy the following equation:

\[
\left( \frac{\delta_i}{h_i} \right) \left( \frac{\sum V_i}{\sum H_i} \right) \leq 0.25
\]

The "sway moments" that result from the first order analysis have to be increased to take into account the additional effects due to the "sway" behaviour of the frame.

For each different storey the value of load factor "\( \alpha_i \)" is obtained with the formula:

\[
\alpha_i = \frac{1}{1 - \left( \frac{\delta_i}{h_i} \right) \left( \frac{\sum V_i}{\sum H_i} \right)}
\]
In our case:

- for load case 2)

LEVEL I
\[ \alpha_i = \frac{1}{1 - 0.149} = 1.175 \]

LEVEL II
\[ \alpha_{ii} = \frac{1}{1 - 0.0229} = 1.023 \]

- for load case 4)

LEVEL I
\[ \alpha_i = \frac{1}{1 - 0.182} = 1.222 \]

LEVEL II
\[ \alpha_{ii} = \frac{1}{1 - 0.0766} = 1.083 \]

For load case 3), according to the condition written above, the amplified moments method should not used. Nevertheless, for the scope of this project, it could be interesting to examine the results for a load case with a strong "sway" behaviour.

- for load case 3)

LEVEL I
\[ \alpha_i = \frac{1}{1 - 0.266} = 1.362 \]

LEVEL II
\[ \alpha_{ii} = \frac{1}{1 - 0.150} = 1.176 \]

According to the EC3, these amplification factors should be used to multiply the "sway moments" (that are those that result from the horizontal loads and also
the vertical loads if either the structure or the loading is asymmetrical). This methodology, as it has been evidenced at point 2.2.7 of the report, *shows to be not applicable in the case of asymmetrical structure* since it not very clear if all the moments that derive from the vertical loading have to be multiplied by the relevant values of "\( \alpha_i \)" or only part of these forces. The alternative method that has been chosen for this specific calculation example (the loads that give the horizontal translation of the storeys are multiplied by the relevant values of "\( \alpha_i \)") is based on the same principle as the one given in the Eurocode; the methodology evidences that the values of the internal forces and moments obtained are conservative to the point that the frame beams and columns sections have to be incremented. Nevertheless, if this method is adopted it is necessary to multiply all the loads (horizontal and vertical) since the frame is asymmetrical.

As a consequence, for each different storey of the frame we obtain the following load cases with increased partial load factors:

- *load case 2)*

**LEVEL I**

\[(1.175 \cdot 1.35) \cdot SWL + (1.175 \cdot 1.35) \cdot DL + (1.175 \cdot 1.35) \cdot IL + (1.175 \cdot 1.35) \cdot FI\_R + (1.175 \cdot 1.35) \cdot WL\]

**LEVEL II**

\[(1.023 \cdot 1.35) \cdot SWL + (1.023 \cdot 1.35) \cdot DL + (1.023 \cdot 1.35) \cdot IL + (1.023 \cdot 1.35) \cdot FI\_R + (1.023 \cdot 1.35) \cdot W\]

- *load case 4)*

**LEVEL I**

\[(1.222 \cdot 1.35) \cdot SWL + (1.222 \cdot 1.35) \cdot DL + (1.222 \cdot 1.35) \cdot IL + (1.222 \cdot 1.35) \cdot FI\_L + (1.222 \cdot 1.35) \cdot W\]

**LEVEL II**

\[(1.083 \cdot 1.35) \cdot SWL + (1.083 \cdot 1.35) \cdot DL + (1.083 \cdot 1.35) \cdot IL + (1.083 \cdot 1.35) \cdot FI\_L + (1.083 \cdot 1.35) \cdot W\]
- load case 3)

LEVEL I

\[(1.362 \cdot 1.35) \cdot SWL + (1.362 \cdot 1.35) \cdot DL + (1.362 \cdot 1.35) \cdot IL + (1.362 \cdot 1.35) \cdot FIL_1 L\]

LEVEL II

\[(1.176 \cdot 1.35) \cdot SWL + (1.176 \cdot 1.35) \cdot DL + (1.176 \cdot 1.35) \cdot IL + (1.176 \cdot 1.35) \cdot FIL_1 L\]

With these new load factors for load cases 2), 3) and 4) a new first order analysis was performed. The results of this calculation are then used in the local member and cross-sectional verifications in which we adopt the "non sway mode" buckling lengths.

In tables 4 and 5 it is possible to compare the internal forces of:
- Rigorous second order method
- Moment amplification method (with "\(\alpha\)" applied to the loads that give the "sway moments").
- First order method

for COLUMNS

- Rigorous second order method
- Moment amplification method (with "\(\alpha\)" applied to the loads that give the "sway moments").
- Sway mode buckling lengths method
- First order method

for BEAMS

The comparison between the two indirect second order methods for the columns is only possible comparing the utilisation factors (expressed as a percentage) that are obtained from the bending-axial force buckling member checks.

In figures 6 to 13, for load cases 2 and 4, are given the diagrams of the bending moments for the different design methods together with the frame deflected shape.

The following remarks can be made:

columns - if we examine the values of the bending moments, it is visible that those that are calculated with a rigorous II order analysis are just slightly higher than those of the I order calculation.
In addition, the "moment amplification" calculation gives values for the moments that are much higher than those that derive from the II order analysis, and thus the method results very conservative.

Once the internal forces and moments have been calculated taking into account (when necessary) the second order effects, it is possible to carry out the local member and cross-sectional checks of the elements of the frame. In this calculation example, only the checks of one column and one beam are illustrated extensively. For the other elements, the computer code developed by A.C.A.I. (Associazione Italiana Costruttori Acciaio) "Eurocode 3", has been used.

The members taken into account, together with their internal forces, are:

- **Column**

  Element 1 (HEA 200), node 2 - Moment amplif. method - load case 4
  
  \[ N = 316 \text{ kN} \]
  \[ V = 14.6 \text{ kN} \]
  \[ M = 55.4 \text{ kN}\cdot\text{m} \]

- **Beam**

  Element 7 (IPE 360), node 2 - Moment amplif. method - load case 4
  
  \[ V = 171 \text{ kN} \]
  \[ M = 138 \text{ kN}\cdot\text{m} \]

It is necessary to perform a preliminary operation: the classification of the cross sections of these members. Since an elastic analysis has been performed, the member cross sections may only be considered to be **class 3 or 4** in the local member and cross sectional checks.

**VII - CLASSIFICATION OF CROSS SECTIONS**
A) Column

HEA 200, \( f_y = 355 \) daN/cm\(^2\)

1) Web

The formulas for pure compression can be taken into account as a simplification:

- **class 1** \( \frac{d}{t_w} \leq 33 \cdot \varepsilon = 33 \cdot 0.814 = 26.9 \)
- **class 2** \( \frac{d}{t_w} \leq 38 \cdot \varepsilon = 38 \cdot 0.814 = 30.9 \)
- **class 3** \( \frac{d}{t_w} \leq 42 \cdot \varepsilon = 42 \cdot 0.814 = 34.2 \)

In our case \( \frac{d}{t_w} = \frac{134}{6.5} = 20.6 \Rightarrow \text{class 1} \)

2) Flange

- **class 1** \( \frac{c}{t_f} \leq 10 \cdot \varepsilon = 10 \cdot 0.814 = 8.14 \)
- **class 2** \( \frac{c}{t_f} \leq 11 \cdot \varepsilon = 11 \cdot 0.814 = 8.95 \)
class 3 \quad \frac{c}{t_f} \leq 15 \cdot \varepsilon = 15 \cdot 0.814 = 12.2

in our case \quad \frac{c}{t_f} = \frac{100}{10} = 10 \implies \text{class 3}

The column can be classified as a class 3 element.

B) Beam

IPE 360, $f_y = 355$ daN/cm²

1) Web

The formulas for pure bending can be taken into account as a simplification:

class 1 \quad \frac{d}{t_w} \leq 72 \cdot \varepsilon = 72 \cdot 0.814 = 58.6

class 2 \quad \frac{d}{t_w} \leq 83 \cdot \varepsilon = 83 \cdot 0.814 = 67.6

class 3 \quad \frac{d}{t_w} \leq 124 \cdot \varepsilon = 124 \cdot 0.814 = 101

in our case \quad \frac{d}{t_w} = \frac{298.6}{8} = 37.3 \implies \text{class 1}

2) Flange

\begin{align*}
\text{class 1} \quad & \frac{c}{t_f} \leq 10 \cdot \varepsilon = 10 \cdot 0.814 = 8.14 \\
\text{class 2} \quad & \frac{c}{t_f} \leq 11 \cdot \varepsilon = 11 \cdot 0.814 = 8.95 \\
\text{class 3} \quad & \frac{c}{t_f} \leq 15 \cdot \varepsilon = 15 \cdot 0.814 = 12.2
\end{align*}
in our case \[ \frac{c}{t_f} = \frac{85}{12.7} = 6.69 \Rightarrow \text{class 1} \]

The beam can be classified as a class 1 element, but since this is an elastic analysis, we have to perform the checks according to class 3 rules.

**EC3**

**VIII - ULS MEMBER AND CROSS SECTION CHECKS**

sectors 5.4 - 5.5

**A) COLUMN**

section 5.4.5  **1) Moment cross sectional resistance**

For class 3 cross sections:

clause 5.4.5.2 \[ M_{c,\text{rd}} = \frac{W_e \cdot f_y}{\gamma_{M0}} \]

In our case:

\[ M_{c,\text{rd}} = \frac{(388.6 \cdot 10^3) \cdot 355 \cdot 10^3}{1.1} = 125.4 > 55.4 \text{ kN*m} \]

section 5.4.6  **2) Shear cross sectional resistance**
clause 5.4.6(1)\[ p_{k,Rd} = \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} \]

where:

clause 5.4.6(2)\[ A_v = A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f \]

for I rolled sections loaded parallel to web.

In our case:

\[ p_{k,Rd} = \frac{[53.83 - 2 \cdot 20 \cdot 1 + (0.65 + 2 \cdot 1.8) \cdot 1] \cdot 355 \cdot 10^{-1}}{\sqrt{3} \cdot 1.1} = 336.9 > 14.6 \text{ kN} \]

section 5.4.4 3) Compression cross sectional resistance

For class 3 cross sections:

clause 5.4.4(2)\[ N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \]

In our case:

\[ N_{c,Rd} = \frac{53.83 \cdot (355 \cdot 10^{-1})}{1.1} = 1737 > 316 \text{ kN} \]

section 5.4.7 4) Bending and shear cross section resistance

clause 5.4.7(2) It is not necessary to take into account any type of iteration between M and V since, in our case, we have:

\[ \frac{V_{sd}}{V_{\mu,Rd}} = \frac{14.6}{336.9} = .0433 < 0.5 \]

section 5.4.8 5) Bending and compression cross section resistance

For class 3 sections, the following formula has to be fulfilled:

clause 5.4.8.2\[ \frac{N_{sd}}{A \cdot \left( \frac{f_y}{\gamma_{M0}} \right)} + \frac{M_{sd}}{W_{et} \left( \frac{f_y}{\gamma_{M0}} \right)} \leq 1 \]

In our case:
section 5.4.9 6) **Bending, shear and compression cross section resistance**

It is not necessary to take into account the iteration between M, V, and N since:

\[ \frac{316}{53.83 \cdot \left(355 \cdot 10^3 \right)^{\frac{1}{11}}} + \frac{55.4}{(388.6 \cdot 10^{-6}) \cdot \left(355 \cdot 10^3 \right)^{\frac{1}{11}}} = 0.6236 \leq 1 \]

section 5.6 7) **Shear buckling resistance**

This calculation does not have to be performed since:

\[ \frac{d}{t_w} = \frac{134}{6.5} = 20.6 \leq 0.814 \cdot 69 = 56.2 \]

section 5.5.4 8) **Bending and compression buckling resistance**

For members with class 3 cross sections the following formula has to be fulfilled:

\[ \frac{N_{sd}}{\chi \cdot A \left(\frac{f_y}{Y_{M1}}\right)} + \frac{k \cdot M_{sd}}{W_{el} \left(\frac{f_y}{Y_{M1}}\right)} \leq 1 \]

where:

\[ \chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} \quad \text{but} \quad \chi \leq 1 \]

in which:

\[ \phi = 0.5 \left[ 1 + \alpha \cdot (\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right] \]

\[ \bar{\lambda} = \frac{\lambda}{93.9 \cdot \frac{235}{f_y} \sqrt{\beta_A}} \]

in which:

\[ \lambda = \frac{i}{i} \quad \text{column slenderness} \]

\[ \beta_A = 1 \]

\[ \alpha = \text{imperfection factor} \]
\[ k = 1 - \frac{\mu \cdot N_{sd}}{\lambda \cdot A \cdot f_y} \quad \text{but} \quad k \leq 1.5 \]

in which:
\[ \mu = \bar{\lambda} \cdot (2 \cdot \beta_M - 4) \quad \text{but} \quad \mu \leq 0.9 \]

where:
\[ \beta_M = \text{equivalent uniform moment factor} \]

The buckling lengths "1" have been calculated, both for the "sway" and the "non sway" mode, by the Lotus 1-2-3 spreadsheet given in tables 6 and 7. These calculations are a simple application of Annex E of the EC3.

In this example, since the internal forces have been calculated using the "indirect second order moment amplification method", the "non sway" buckling lengths have been adopted.

In our case:

\[ l = L \cdot \beta \]
\[ = 380 \cdot 0.855 = 325 \text{ cm} \]
\[ \lambda = \frac{325}{8.28} = 39.25 \]
\[ \beta_M = 1.8 \]
\[ \lambda = \frac{39.25}{93.9 \cdot \frac{235}{355}} \cdot \sqrt{I} = 0.5138 \]
\[ \mu = 0.5138 \cdot (2 \cdot 1.8 - 4) = -0.2055 \]
\[ \alpha = 0.34 \quad \text{(buckling curve "b")} \]
\[ \phi = 0.5 \left[ 1 + 0.34 \cdot (0.5138 - 0.2) + 0.5138^2 \right] = 0.6853 \]
\[ \chi = \frac{1}{0.6853 + \sqrt{0.6853^2 - 0.5138^2}} = 0.878 \]
\[ k = 1 - \frac{-0.2055 \cdot 316}{0.8781 \cdot 53.83 \cdot (355 \cdot 10^{-1})} = 1.039 \]

and thus:
\[ \frac{316}{0.8781 \cdot 53.83 \cdot (355 \cdot 10^{-1})} + \frac{1.039 \cdot 5540}{388.6 \cdot (355 \cdot 10^{-1})} = 0.6661 \leq 1 \]

In table 8 it is possible to compare the utilisation ratios, for the M-N buckling check, of some columns calculated with the two indirect second order methods.
given in the EC3; the "moment amplification method" and the "sway buckling lengths method".
The "sway buckling lengths method is more conservative than the "moment amplification method" for the columns of the first storey; for those of the second level the situation is completely different and the first method results to be more conservative.

B) BEAM

section 5.4.5 1) Moment cross sectional resistance

For class 3 cross sections:
clause 5.4.5.2
\[ M_{c,Rd} = \frac{W_e \cdot f_y}{\gamma_{M0}} \]

In our case:
\[ M_{c,Rd} = \frac{(903.6 \cdot 10^{-6}) \cdot (355 \cdot 10^3)}{1.1} = 291.6 > 138 \text{ kN*m} \]

section 5.4.6 2) Shear cross sectional resistance

clause 5.4.6(1)
\[ p_{l,Rd} = \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} \]

where:
clause 5.4.6(2)
\[ A_v = A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f \]
for I rolled sections loaded parallel to web

In our case:
\[ p_{l,Rd} = \sqrt{3} \cdot 1.1 \]
\[ = 654.7 > 171 \text{ kN} \]

section 5.4.7 3) Bending and shear cross section resistance

clause 5.4.7(2) It is not necessary to take into account any type of iteration between M and V since, in our case, we have:
\[ \frac{V_{rd}}{V_{p,l,Rd}} = \frac{171}{654.7} = 0.261 < 0.5 \]
section 5.6 4) Shear buckling resistance

This calculation does not have to be performed since:

\[
\frac{d}{t_w} = \frac{298.6}{8} = 37.3 \leq 0.814 \cdot 69 = 56.2
\]

IX - BEAM - COLUMN CONNECTION CHECKS

The calculation of these beam-column "end plate connections" is based on a simplified procedure adopting Annex J of the EC3. The complete design procedure J.3.1 has not been adopted since it will soon be replaced by the rules given in annex JJ.

A) CONNECTING ELEMENTS

- Beam : IPE 360, Fe 510
- Column : HEA 200, Fe 510
- End Plate : 440x200x16, Fe 510
- Web Plate : 800x122x8, Fe 510
- Bolts : 8 M20, 8.8

In fig.4 it represented a lateral view of the connection.

B) LOADS

Design loads:
Msd = 138 kN*m
Vsd = 171 kN

EC3 - Annex J
section J.3

C) CALCULATION OF THE MOMENT RESISTANCE

The calculation is performed dividing the connection in zones, and performing an independent check for each one.

1) Tension zone

clause J.3.4.3

a) Stiffened column flange

The "equivalent T stub" theory is adopted.

The total "T stub" length is determined from the effective length of the bolt pattern:

For one row of bolts adjacent to a stiffener:

\[ l_{\text{eff},a} = \text{MIN}(\alpha \cdot m, 2 \cdot \pi \cdot m) \]

in which:

\[ m = \frac{b}{2} - e - 0.8 \cdot r - \frac{t_w}{2} \]

\[ e = \text{bolts horizontal edge distance} \]

\[ \alpha = \text{ratio depending on the position of the bolts in relation to the stiffener} \]

\[ m_2 = z - 0.8 \cdot \sqrt{2} \cdot a_{\text{eff}} \]

In our case:

\[ e = 40 \text{ mm} \]

\[ m = \frac{200}{2} - 40 - 0.8 \cdot 18 - \frac{6.5}{2} = 42.4 \text{ mm} \]

\[ m_2 = 36 - 0.8 \cdot \sqrt{2} \cdot 6 = 29.2 \text{ mm} \]

Fig. J.3.7

For the calculation of \( \alpha \):

\[ \lambda_1 = \frac{m}{m + e} = \frac{42.4}{42.4 + 40} = 0.515 \]

\[ \lambda_2 = \frac{m_2}{m + e} = \frac{29.2}{42.4 + 40} = 0.354 \]

\[ \alpha = 6.5 \text{ (supposed)} \]

\[ l_{\text{eff},a} = \text{MIN}(6.5 \cdot 42.4, 2 \cdot \pi \cdot 42.4) = 266.4 \text{ mm} \]
section J.3.3 Calculation of the plate bending moment of the "T stub":

\[ M_{pl,Rd} = 0.25 \cdot I_{eff,a} \cdot t \cdot f_y \cdot k_r / \gamma_{M0} \]

In our case:

\[ M_{pl,Rd} = \left[ 0.25 \cdot 266.4 \cdot 10^2 \cdot \frac{355}{1.1} \right] \cdot 10^{-6} = 2.15 \text{ kN*m} \]

clause J.3.3 (3) Determination of the failure mode of the "T stub":

\[ \beta = \frac{4 \cdot M_{pl,Rd}}{m \cdot \sum B_{i,Rd}} \]

in which:

\[ B_{i,Rd} = \frac{0.9 \cdot f_{ub} \cdot A_s}{\gamma_{ Mb}} \]

\[ \lambda = \frac{e}{m} \]

In our case:

\[ \lambda = \frac{40}{42.4} = 0.943 \quad \text{and thus} \quad \frac{2 \cdot \lambda}{1 + 2 \cdot \lambda} = \frac{2 \cdot 0.943}{1 + 2 \cdot 0.943} = 0.653 \]

\[ \beta = \frac{4 \cdot 2.15}{(42.4 \cdot 10^{-3}) \cdot 2 \cdot 141.1} = 0.719 \]

Fig. J.3.3 0.653 < \beta < 2 \text{ failure mode 2}

The design tension resistance of the two "T stubs" (one in each side of the stiffener) can now be calculated:

\[ F_{t,Rd} = 2 \cdot \left[ \frac{2 \cdot M_{pl,Rd} + e \cdot \sum B_{i,Rd}}{m + e} \right] = 2 \cdot \left[ \frac{2 \cdot 2.15 + (40 \cdot 10^{-3}) \cdot 2 \cdot 141.1}{(42.4 + 40) \cdot 10^{-3}} \right] = 398.3 \text{ kN} \]

clause J.3.4.4 b) End plate

The "equivalent T stub" theory is adopted.
The total "T stub" length is determined from the effective length of the bolt pattern:

- For bolts outside the tension flange of the beam (extension "T stub"):
  \[ l_{\text{eff},a} = \text{MIN}(0.5 \cdot b_p, 0.5 \cdot w + 2 \cdot m_x + 0.625 \cdot e_x, 4 \cdot m_x + 1.25 \cdot e_x, 2 \cdot \pi \cdot m) \]

- For first row of bolts below tension flange of beam (inner "T stub"):
  \[ l_{\text{eff},b} = \text{MIN}(\alpha \cdot m, 2 \cdot \pi \cdot m) \]

in which:
- \( b_p \) = plate width
- \( w \) = bolts horizontal spacing
- \( m_x \) = vertical distance between tension beam flange and outer row of bolts
- \( e_x \) = bolts vertical edge distance

\[ m = \frac{b_p}{2} - e - 0.8 \cdot \sqrt{2} \cdot \alpha - \frac{t_x}{2} \]

\( \alpha \) = ratio depending on the position of the bolts in relation to the stiffener

In our case:

\[ e = 40 \text{ mm} \]
\[ m = \frac{200}{2} - 40 - 0.8 \cdot \sqrt{2} \cdot 6 - \frac{8}{2} = 49.2 \text{ mm} \]
\[ m_x = (120 - 70 - 12.7) - 0.8 \cdot \sqrt{2} \cdot 6 = 30.5 \text{ mm} \]

For the calculation of \( \alpha \):

\[ \lambda_1 = \frac{m}{m + e} = \frac{49.2}{49.2 + 40} = 0.552 \]
\[ \lambda_2 = \frac{m_x}{m + e} = \frac{30.5}{49.2 + 40} = 0.342 \]

\[ \alpha = 6.5 \text{ (supposed)} \]
\[ b_p = 200 \text{ mm} \]
\[ w = 120 \text{ mm} \]
\[ m_x = 37 - 0.8 \cdot \sqrt{2} \cdot 6 = 30.2 \text{ mm} \]
\[ e_x = 33 \text{ mm} \]

\[ l_{\text{eff},a} = \text{MIN}(0.5 \cdot 200, 0.5 \cdot 120 + 2 \cdot 30.2 + 0.625 \cdot 33.4 \cdot 30.2 + 1.25 \cdot 33.2 \cdot \pi \cdot 28.5) = 100 \text{ mm} \]

\[ l_{\text{eff},b} = \text{MIN}(6.5 \cdot 49.2, 2 \cdot \pi \cdot 49.2) = 309.1 \text{ mm} \]
section 3.3 Calculation of the plate bending moment of the "T stubs":

\[ M_{pl,Rd1} = 0.25 \cdot 100 \cdot 16^2 \cdot \frac{355}{11} \cdot 10^{-6} = 2.07 \text{ kN} \cdot \text{m} \]  
\text{(extension "T stub")}

\[ M_{pl,Rd2} = 0.25 \cdot 309.1 \cdot 16^2 \cdot \frac{355}{11} \cdot 10^{-6} = 6.38 \text{ kN} \cdot \text{m} \]  
\text{(inner "T stub")}

clause 3.3 (3) Determination of the failure mode of the "T stubs":

\[ \beta = \frac{4 \cdot M_{pl,Rd}}{m \cdot \sum B_{i,Rd}} \]

in which:

\[ B_{i,Rd} = \frac{0.9 \cdot f_{ub} \cdot A_s}{\gamma_{mf}} \]

\[ \lambda = \frac{e}{m} \]

In our case:

- for the \textit{extension "T stub"}

\[ \lambda = \frac{33}{30.2} = 1.093 \]

and thus

\[ \frac{2 \cdot \lambda}{1 + 2 \cdot \lambda} = \frac{2 \cdot 1.093}{1 + 2 \cdot 1.093} = 0.686 \]

\[ \beta = \frac{4 \cdot 2.07}{(30.2 \cdot 10^{-3}) \cdot 2 \cdot 141.1} = 0.972 \]

\text{Fig. 3.3} 0.686 < \beta < 2 \text{ failure mode 2}

- for the \textit{inner "T stub"}

\[ \lambda = \frac{40}{49.2} = 0.813 \]

and thus

\[ \frac{2 \cdot \lambda}{1 + 2 \cdot \lambda} = \frac{2 \cdot 0.813}{1 + 2 \cdot 0.813} = 0.619 \]

\[ \beta = \frac{4 \cdot 6.38}{(49.2 \cdot 10^{-3}) \cdot 2 \cdot 141.1} = 1.83 \]

\text{Fig. 3.3} 0.619 < \beta < 2 \text{ failure mode 2}

The design tension resistance of each one of the two "T stubs" can be calculated and summed:
\[ F_{t,Rd} = \frac{2 \cdot M_{pl,Rd} + e \cdot \sum B_{i,Rd}}{m + e} \]

In our case:

\[ F_{t,Rd1} = \frac{2 \cdot 2.07 \cdot 10^3 + 33 \cdot 2.141.1}{30.2 + 33} = 212.9 \text{ kN} \]

\[ F_{t,Rd2} = \frac{2 \cdot 6.38 \cdot 10^3 + 40 \cdot 2.141.1}{40 + 49.2} = 269.6 \text{ kN} \]

\[ F_{t,Rd} = 212.9 + 269.6 = 482.5 \text{ kN} \]

clause 1.3.4.6  

**c) Column web**

The design resistance of the column web is given by:

\[ F_{t,Rd} = \frac{f_{yc} \cdot t_{wc,eff} \cdot b_{eff}}{Y_{M0}} \]

In which:

clause 1.2.3.2(4) \( t_{wc,eff} = 1.4 \cdot t_{wc} \)

clause 1.3.4.6(2) \( b_{eff} = 2 \cdot l_{eff,a} \)

In our case:

\( t_{wc,eff} = 1.4 \cdot 6.5 = 9.1 \text{ mm} \)

\( b_{eff} = 2 \cdot 266.4 = 532.8 \text{ mm} \)

\[ F_{t,Rd} = \frac{355 \cdot 9.1 \cdot 532.8}{1.1} \cdot 10^{-3} = 1565 \text{ kN} \]

**2) Compression zone**

section J.3.5.1  

**a) Column web**

The design crushing resistance of the column web subject to a transverse compression force is given by:

\[ F_{c,Rd} = \frac{f_{yc} \cdot t_{wc,eff} \cdot b_{eff}}{Y_{M0}} \]

In which:
\[ b_{\text{eff}} = t_p + 2\sqrt{2} \cdot a_p + 2 \cdot t_p + 5 \cdot (f_c + r_c) \]
\[ t_{wceff} = 1.5 \cdot t_w \]

In our case:
\[ b_{\text{eff}} = 12.7 + 2\sqrt{2} \cdot 6 + 2 \cdot 16 + 5 \cdot (10 + 18) = 201.7 \text{ mm} \]
\[ t_{wceff} = 1.5 \cdot 6.5 = 9.75 \text{ mm} \]
\[ F_{e,\text{lt}} = \frac{355 \cdot 9.75 \cdot 201.7}{1.1} \cdot 10^{-3} = 634.7 \text{ kN} \]

section J.3.6

3) Shear zone

a) Column web

clause 5.6.1(1) A check to verify if a shear buckling calculation is necessary or not:
\[ \frac{d}{t_w} = \frac{134}{6.5} = 20.6 < 69 \cdot 0.814 = 56.2 \]

the check is not necessary

The design plastic shear resistance of the column web strengthened by a supplementary web panel is:

\[ \varphi_{\text{pl},Rd} = \frac{f_{x e} \cdot A_v}{\sqrt{3} \cdot \gamma_{M0}} \]

where:
\[ A_v = A_{v,\text{col}} + A_{v,wp} \]
\[ A_{v,\text{col}} = A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) t_f \]
\[ A_{v,wp} = b_{wp} \cdot t_{wp} \]

In our case:
\[ A_{v,\text{col}} = 53.8 - 2 \cdot 20 \cdot 1 + (0.65 + 2 \cdot 18) \cdot 1 = 18.1 \text{ cm}^2 \]
\[ A_{v,wp} = 12.2 \cdot 0.8 = 9.76 \text{ cm}^2 \]
\[ V_{\varphi,\text{pl},Rd} = \frac{355 \cdot (18.1 + 9.76) \cdot 10^{-3}}{\sqrt{3} \cdot 1.1} = 519.1 \text{ kN} \]

section J.3.6

4) Final check
The moment resistance, taking into account the resistances of the three zones, is calculated taking into account the minimum value of the elementary resistances:

\[ F_{Rd} = 398.3 \text{ KN} \]

\[ M_{Rd} = 398.3 \times (360 - 2 \times 12.7 / 2) \times 10^{-3} \]

\[ = 138.3 > 138.0 \text{ KN} \times \text{m} \quad \text{The check is fulfilled} \]

**D) CALCULATION OF THE SHEAR RESISTANCE**

1) Shear resistance

We assume that only the lower four bolts transfer the shear force from the beam to the column:

\[ F_{V,sd} = \frac{171}{4} = 42.8 \text{ kN} \]

\[ F_{V,Rd} = \frac{0.6 \cdot f_{ub} \cdot A_s}{\gamma_{Mb}} = 94.1 \text{ kN} \]

\[ F_{V,Rd} > F_{V,sd} \quad \text{The check is fulfilled} \]

2) Column flange bearing resistance

We assume that only the lower four bolts transfer the shear force from the beam to the column:

\[ F_{V,sd} = \frac{171}{4} = 42.8 \text{ kN} \]

\[ F_{V,Rd} = \frac{2.5 \cdot \alpha \cdot f_u \cdot d \cdot t}{\gamma_{Mb}} \]

where:

\[ \alpha = \text{MIN} \left[ \frac{e_1}{3 \cdot d_0}, \frac{P_1}{3 \cdot d_0}, 0.25 \frac{f_{ub}}{f_u}, 1 \right] \]

In our case:
- \( e_1 = \) not specified
- \( P_1 = 126 \text{ mm} \)
- \( d_0 = 22 \text{ mm} \)
- \( f_{ub} = 800 \text{ N/mm}^2 \)
- \( f_u = 510 \text{ N/mm}^2 \)
\[ \alpha = \text{MIN} \left[ -\frac{126}{3 \cdot 22}, -0.25, \frac{800}{510}, 1 \right] = 1 \]

\[ F_{v,Rd} = \frac{2.5 \cdot 1 \cdot 510 \cdot 20 \cdot 10}{1.25} \cdot 10^{-3} = 204 \text{ kN} \]

\[ F_{v,Rd} > F_{v,Sd} \quad \text{The check is fulfilled} \]
Civil building (CIV) analysis
Frame structural model

Note: the dimensions are indicated in millimetres

ECSC Project 7210-5A/419
Practical application of EUROCODE 3
to multi-storey buildings with steel
"Sway Frame" structures
Civil building (CIV) analysis

Building units arrangement

Note: the dimensions are indicated in metres

ECSC Project 7210-SA/419
Practical application of EUROCODE 3 to multi-storey buildings with steel "Sway Frame" structures
Civil building (CIV) non seismic analysis

Member profiles

IPE 360  IPE 300

K.

IPE 360

CM <

Ld X

J

K.

IPE 300

XI <

LU X

y?

ECSC Project 7210-SA/419

Practical application of EUROCODE 3 to multi-storey buildings with steel “Sway Frame” structures
SECTION 1–1

Note: the dimensions are indicated in millimetres

ECSC Project 7210-SA/419
Practical application of EUROCODE 3 to multi-storey buildings with steel "Sway Frame" structures
FIGURE 5
Stresses acting on the Structure elev_t0 (Moment y)
Analysis Number 1 - Load Combination Number 2

FIGURE 6
FIRST ORDER - AMPLIFIED SWAY MOMENTS
(amplification factors applied to loads)
Stresses acting on the Structure clv_t0 (Moment y)
Analysis Number 1 - Load Combination Number 2

FIGURE 8
Sidercad S.p.A - Stranger - Post Analysis
Displacements of the Structure clv_tO
Analysis Number 1 - Load Combination Number 2
2 ULS

FIGURE 9
Strains acting on the structure clv_t0 (Moment y)
Analysis Number 1 - Load Combination Number 4

FIGURE 10
Stresses acting on the Structure clv_t0 (Moment y)

Analysis Number 1 - Load Combination Number 4

FIRST ORDER - AMPLIFIED SWAY MOMENTS
(amplification factors applied to loads)

FIGURE 11
SECOND ORDER

Stresses acting on the Structure clv_t0 (Moment y)
Analysis Number 1 - Load Combination Number 4

FIGURE 12
FIGURE 13
### ECSC Project SA/419 - "EC3 Sway Frames"  
**EUROCODE 1: WIND ACTIONS**

**Title:** Civil building (CIV) calculation example  
**Date:**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>C_t</td>
<td>1</td>
</tr>
<tr>
<td>I</td>
<td>40.6 (m)</td>
</tr>
<tr>
<td>C_DIR</td>
<td>1</td>
</tr>
<tr>
<td>C_TEM</td>
<td>1</td>
</tr>
<tr>
<td>a_s</td>
<td>0 (Km)</td>
</tr>
<tr>
<td>K_a</td>
<td>0.55 (1/Km)</td>
</tr>
<tr>
<td>Vref,0</td>
<td>28 (m/s)</td>
</tr>
<tr>
<td>B</td>
<td>5 (m)</td>
</tr>
<tr>
<td>K_r</td>
<td>0.24</td>
</tr>
<tr>
<td>z₀</td>
<td>1 (m)</td>
</tr>
<tr>
<td>z_min</td>
<td>16 (m)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Level</th>
<th>height building portion</th>
<th>reference elevation</th>
<th>positive pressure coefficient</th>
<th>negative pressure coefficient</th>
<th>positive storey load</th>
<th>negative storey load</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>3.5</td>
<td>3.8</td>
<td>0.8</td>
<td>0.3</td>
<td>7.53</td>
<td>2.83</td>
</tr>
<tr>
<td>II</td>
<td>1.6</td>
<td>7</td>
<td>0.8</td>
<td>0.3</td>
<td>3.44</td>
<td>1.29</td>
</tr>
<tr>
<td>III</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>IV</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>V</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>VI</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>VII</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>VIII</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>IX</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**TABLE 1**
### ECSC Project SA/419 - "EC3 Sway Frames" EUROCODE 3: SWAY - NON SWAY CRITERIA

**Title:** Civil building (CV) - Non seismic design

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N  (KN)</th>
<th>V  (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
<td>263</td>
<td>-5.770</td>
<td>0.000</td>
<td>2</td>
<td>0.093</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>589</td>
<td>-6.120</td>
<td></td>
<td>3</td>
<td>0.115</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>189</td>
<td>-3.800</td>
<td></td>
<td>8</td>
<td>0.115</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Storey total forces (KN)</td>
<td>1041</td>
<td>-4.150</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Storey height</td>
<td>380</td>
<td>mean rel. disp. (cm)</td>
</tr>
</tbody>
</table>

**Verifica sway:** 
\[
\frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}}} = 0.0587
\]

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N  (KN)</th>
<th>V  (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1</td>
<td>2</td>
<td>130</td>
<td>41.600</td>
<td>0.093</td>
<td>3</td>
<td>0.027</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>298</td>
<td>-24.300</td>
<td></td>
<td>6</td>
<td>-0.114</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>92</td>
<td>-19.400</td>
<td>0.115</td>
<td>9</td>
<td>-0.114</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Storey total forces (KN)</td>
<td>520</td>
<td>-2.100</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Storey height</td>
<td>320</td>
<td>mean rel. disp. (cm)</td>
</tr>
</tbody>
</table>

**Verifica sway:** 
\[
\frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}}} = 0.1485
\]

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N  (KN)</th>
<th>V  (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>1</td>
<td>254</td>
<td>1.360</td>
<td>0.000</td>
<td>2</td>
<td>1.321</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>590</td>
<td>-19.100</td>
<td></td>
<td>4</td>
<td>0.312</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>197</td>
<td>-6.780</td>
<td>0.000</td>
<td>6</td>
<td>1.339</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Storey total forces (KN)</td>
<td>1041</td>
<td>-24.320</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Storey height</td>
<td>380</td>
<td>mean rel. disp. (cm)</td>
</tr>
</tbody>
</table>

**Verifica sway:** 
\[
\frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}}} = 0.120
\]

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N  (KN)</th>
<th>V  (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>2</td>
<td>2</td>
<td>128</td>
<td>38.700</td>
<td>1.321</td>
<td>3</td>
<td>1.465</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>298</td>
<td>-25.800</td>
<td></td>
<td>6</td>
<td>0.144</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>94</td>
<td>-21.400</td>
<td>1.339</td>
<td>9</td>
<td>0.096</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Storey total forces (KN)</td>
<td>520</td>
<td>-8.500</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Storey height</td>
<td>320</td>
<td>mean rel. disp. (cm)</td>
</tr>
</tbody>
</table>

**Verifica sway:** 
\[
\frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}}} = 0.0229
\]

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N  (KN)</th>
<th>V  (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3</td>
<td>1</td>
<td>267</td>
<td>7.580</td>
<td>0.000</td>
<td>2</td>
<td>0.415</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>589</td>
<td>-0.843</td>
<td>0.000</td>
<td>5</td>
<td>0.393</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>186</td>
<td>-2.580</td>
<td>0.000</td>
<td>8</td>
<td>0.393</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Storey total forces (KN)</td>
<td>1042</td>
<td>4.157</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Storey height</td>
<td>380</td>
<td>mean rel. disp. (cm)</td>
</tr>
</tbody>
</table>

**Verifica sway:** 
\[
\frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}}} = 0.2665
\]

**TABLE 2**
<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2 3</td>
<td>2</td>
<td>131</td>
<td>43.200</td>
<td>0.415</td>
<td>3</td>
<td>0.585</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>298</td>
<td>-22.800</td>
<td>0.393</td>
<td>6</td>
<td>0.611</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>91</td>
<td>-18.300</td>
<td>0.393</td>
<td>9</td>
<td>0.218</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>520</td>
<td>2.100</td>
<td>Storey height (cm)</td>
<td>320</td>
<td>mean rel. disp. (cm)</td>
<td>0.194</td>
<td></td>
</tr>
<tr>
<td>Verifica sway:</td>
<td>(N tot x st.disp.) / (H x V tot)=</td>
<td>0.1502</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 4</td>
<td>1</td>
<td>276</td>
<td>12.000</td>
<td>0.000</td>
<td>2</td>
<td>1.640</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>587</td>
<td>12.200</td>
<td>0.000</td>
<td>5</td>
<td>1.640</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>179</td>
<td>0.410</td>
<td>0.000</td>
<td>8</td>
<td>1.619</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>1042</td>
<td>24.610</td>
<td>Storey height (cm)</td>
<td>380</td>
<td>mean rel. disp. (cm)</td>
<td>1.630</td>
<td></td>
</tr>
<tr>
<td>Verifica sway:</td>
<td>(N tot x st.disp.) / (H x V tot)=</td>
<td>0.1816</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2 4</td>
<td>2</td>
<td>133</td>
<td>46.200</td>
<td>1.640</td>
<td>3</td>
<td>2.015</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>298</td>
<td>-21.400</td>
<td>1.619</td>
<td>6</td>
<td>0.426</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>90</td>
<td>-16.300</td>
<td>1.619</td>
<td>9</td>
<td>0.400</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>520</td>
<td>8.500</td>
<td>Storey height (cm)</td>
<td>320</td>
<td>mean rel. disp. (cm)</td>
<td>0.400</td>
<td></td>
</tr>
<tr>
<td>Verifica sway:</td>
<td>(N tot x st.disp.) / (H x V tot)=</td>
<td>0.0765</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>0</td>
<td>0.000</td>
<td>Storey height (cm)</td>
<td>mean rel. disp. (cm)</td>
<td>0.000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Verifica sway:</td>
<td>(N tot x st.disp.) / (H x V tot)=</td>
<td>0.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>0</td>
<td>0.000</td>
<td>Storey height (cm)</td>
<td>mean rel. disp. (cm)</td>
<td>0.000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Verifica sway:</td>
<td>(N tot x st.disp.) / (H x V tot)=</td>
<td>0.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 3**
<table>
<thead>
<tr>
<th>Elem. Num.</th>
<th>Node Num.</th>
<th>Comb. Num.</th>
<th>I order N (kN)</th>
<th>I order V (kN)</th>
<th>I order M (kN*m)</th>
<th>I order - Amplified moments N (kN)</th>
<th>I order - Amplified moments V (kN)</th>
<th>I order - SwaY mode buck. length M (kN*m)</th>
<th>II order N (kN)</th>
<th>II order V (kN)</th>
<th>II order M (kN*m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 2 3</td>
<td>265 -7.58</td>
<td>28.8</td>
<td>336 -10.5</td>
<td>39.9</td>
<td>265 -7.45</td>
<td>29.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 5 3</td>
<td>584 0.843</td>
<td>-3.20</td>
<td>741 1.60</td>
<td>-6.08</td>
<td>585 0.714</td>
<td>-0.211</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 8 3</td>
<td>184 2.58</td>
<td>-9.80</td>
<td>234 3.63</td>
<td>-13.8</td>
<td>183 2.57</td>
<td>-9.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 3 3</td>
<td>129 -43.2</td>
<td>72.1</td>
<td>153 -54.3</td>
<td>87.9</td>
<td>129 -43.3</td>
<td>72.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 6 3</td>
<td>294 22.8</td>
<td>-38.7</td>
<td>345 28.7</td>
<td>-45.9</td>
<td>294 23.0</td>
<td>-38.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 9 3</td>
<td>89.6 18.3</td>
<td>-30.1</td>
<td>106 23.1</td>
<td>-36.7</td>
<td>89.6 18.2</td>
<td>-29.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 2 4</td>
<td>274 -12.0</td>
<td>45.5</td>
<td>316 -14.6</td>
<td>55.4</td>
<td>276 -11.6</td>
<td>49.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 5 4</td>
<td>583 -12.2</td>
<td>46.2</td>
<td>672 -14.0</td>
<td>53.0</td>
<td>583 -12.6</td>
<td>58.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 8 4</td>
<td>176 -0.410</td>
<td>1.56</td>
<td>203 -0.280</td>
<td>1.06</td>
<td>174 0.320</td>
<td>4.25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 3 4</td>
<td>131 -46.2</td>
<td>77.6</td>
<td>142 -52.7</td>
<td>86.6</td>
<td>131 -46.6</td>
<td>78.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 6 4</td>
<td>294 21.4</td>
<td>-29.0</td>
<td>318 24.8</td>
<td>-31.3</td>
<td>294 22.0</td>
<td>-27.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 9 4</td>
<td>88.1 16.3</td>
<td>-26.4</td>
<td>95.8 18.8</td>
<td>-29.4</td>
<td>87.9 16.1</td>
<td>-25.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## EC3: "SWAY FRAMES" ANALYSIS RESULTS

### INTERNAL FORCES AND MOMENTS

<table>
<thead>
<tr>
<th>Elem. Num.</th>
<th>Node Num.</th>
<th>Comb. Num.</th>
<th>1 order</th>
<th>1 order - Amplified moments</th>
<th>1 order - Sway mode buck. length</th>
<th>II order</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>N [kN]</td>
<td>V [kN]</td>
<td>M [kN*m]</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>2</td>
<td>4</td>
<td>-141</td>
<td>-116</td>
<td>171</td>
<td>-138</td>
</tr>
<tr>
<td>7</td>
<td>5</td>
<td>4</td>
<td>-149</td>
<td>-143</td>
<td>-184</td>
<td>-177</td>
</tr>
<tr>
<td>8</td>
<td>5</td>
<td>4</td>
<td>136</td>
<td>-150</td>
<td>166</td>
<td>-182</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>4</td>
<td>-86.2</td>
<td>-24.2</td>
<td>-105</td>
<td>-29.7</td>
</tr>
<tr>
<td>9</td>
<td>3</td>
<td>4</td>
<td>131</td>
<td>-77.6</td>
<td>142</td>
<td>-86.6</td>
</tr>
<tr>
<td>9</td>
<td>6</td>
<td>4</td>
<td>-160</td>
<td>-171</td>
<td>-172</td>
<td>-184</td>
</tr>
<tr>
<td>10</td>
<td>6</td>
<td>4</td>
<td>134</td>
<td>-142</td>
<td>145</td>
<td>-153</td>
</tr>
<tr>
<td>10</td>
<td>9</td>
<td>4</td>
<td>-88.1</td>
<td>-26.4</td>
<td>-95.8</td>
<td>-29.4</td>
</tr>
</tbody>
</table>

---

**TABLE 5**
## Civil building (CIV) non seismic calculation example

<table>
<thead>
<tr>
<th>COLUMN NUMBER</th>
<th>DEFL. MODE</th>
<th>KC</th>
<th>K1</th>
<th>K2</th>
<th>gA</th>
<th>K_A</th>
<th>KA</th>
<th>gB</th>
<th>K_B</th>
<th>KB</th>
<th>gC</th>
<th>K_C</th>
<th>KC</th>
<th>gD</th>
<th>K_D</th>
<th>KD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>N</td>
<td>9.72</td>
<td>11.5</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.5</td>
<td>25</td>
<td>12.5</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>N</td>
<td>11.5</td>
<td>0</td>
<td>9.72</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.5</td>
<td>25</td>
<td>12.5</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.5</td>
<td>25</td>
<td>12.5</td>
</tr>
<tr>
<td>1</td>
<td>S</td>
<td>9.72</td>
<td>11.5</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1.5</td>
<td>25</td>
<td>37.5</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>S</td>
<td>11.5</td>
<td>0</td>
<td>9.72</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1.5</td>
<td>25</td>
<td>37.5</td>
<td>0</td>
<td>0</td>
<td>1.5</td>
<td>25</td>
<td>37.5</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>N</td>
<td>36</td>
<td>42.7</td>
<td>0</td>
<td>0.5</td>
<td>25</td>
<td>12.5</td>
<td>0.5</td>
<td>16.7</td>
<td>8.35</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1.5</td>
<td>25</td>
<td>37.5</td>
</tr>
<tr>
<td>4</td>
<td>N</td>
<td>42.7</td>
<td>0</td>
<td>36</td>
<td>0.5</td>
<td>25</td>
<td>12.5</td>
<td>0.5</td>
<td>16.7</td>
<td>8.35</td>
<td>0.5</td>
<td>25</td>
<td>12.5</td>
<td>0.5</td>
<td>16.7</td>
<td>8.35</td>
</tr>
<tr>
<td>3</td>
<td>S</td>
<td>36</td>
<td>42.7</td>
<td>0</td>
<td>1.5</td>
<td>25</td>
<td>37.5</td>
<td>1.5</td>
<td>16.7</td>
<td>25.0</td>
<td>1.5</td>
<td>25</td>
<td>37.5</td>
<td>1.5</td>
<td>16.7</td>
<td>25.0</td>
</tr>
<tr>
<td>4</td>
<td>S</td>
<td>42.7</td>
<td>0</td>
<td>36</td>
<td>1.5</td>
<td>25</td>
<td>37.5</td>
<td>1.5</td>
<td>16.7</td>
<td>25.0</td>
<td>1.5</td>
<td>25</td>
<td>37.5</td>
<td>1.5</td>
<td>16.7</td>
<td>25.0</td>
</tr>
<tr>
<td>5</td>
<td>N</td>
<td>6.61</td>
<td>7.84</td>
<td>0</td>
<td>0.5</td>
<td>16.7</td>
<td>8.35</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.5</td>
<td>16.7</td>
<td>8.35</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>N</td>
<td>7.84</td>
<td>0</td>
<td>6.61</td>
<td>0.5</td>
<td>16.7</td>
<td>8.35</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.5</td>
<td>16.7</td>
<td>8.35</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Values of "g" factors (for beams):
- fixed at far end: 1
- pinned at far end: 0.75
- single curvature: 0.5
- double curvature: 1.5

Values of "n" for each column end:
- fixed end: 0
- pinned end: 1

TABLE 6
<table>
<thead>
<tr>
<th>COLUMN NUMBER</th>
<th>DEFL. MODE</th>
<th>COLUMNS</th>
<th>BEAMS</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>S</td>
<td>KC 6.61</td>
<td>K1 7.84</td>
</tr>
<tr>
<td>6</td>
<td>S</td>
<td>KC 7.84</td>
<td>K1 0.61</td>
</tr>
</tbody>
</table>

Values of “g” factors (for beams):
- fixed at far end: 1
- pinned at far end: 0.75
- single curvature: 0.5
- double curvature: 1.5

Values of “n” for each column end:
- fixed end: 0
- pinned end: 1

TABLE 7
<table>
<thead>
<tr>
<th>COLUMN NUMBER</th>
<th>LOAD COMB. RATIO</th>
<th>BENDING-COMPRESSION BUCKLING CHECK RATIO</th>
<th>AMPLIFIED SWAY MOMENTS METHOD</th>
<th>SWAY MODE BUCKLING LENGTHS METHOD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td>0.7950</td>
<td></td>
<td>0.8469</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>0.8268</td>
<td></td>
<td>0.7501</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>0.4654</td>
<td></td>
<td>0.5130</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>0.2256</td>
<td></td>
<td>0.2096</td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>0.2985</td>
<td></td>
<td>0.3913</td>
</tr>
<tr>
<td>6</td>
<td>4</td>
<td>0.4258</td>
<td></td>
<td>0.3873</td>
</tr>
</tbody>
</table>

TABLE 8
CIVIL BUILDING (CIV)

SEISMIC CALCULATION
**E.C.S.C. 7210 SA/419 - EUROCODE 3 "SWAY FRAMES"**

**Type of building** : Civil (CIV)
**Type of design** : Seismic

### BUILDING AND STRUCTURAL DATA

#### Dimensions
- **Length (m)** : 40
- **Width (m)** : 11.5
- **Height (m)** : 7.0

#### Storeys
- **Number** : 2

#### Structural system
- **Transversal direction**: the bracing system for the horizontal loads is given by nine beam-column rigid jointed frames whose distance is 5 m. The columns of these structures are supposed to be hinged at the bases.
- **Longitudinal direction**: the bracing system is obtained with a number of cross bracing structures that are added to the frame that is supposed to have hinged joints.

### MATERIAL
- **Steel Grade (for profiles, plates and welds)** : Fe 510
- **Bolts grade** : 8.8

### SEISMIC DESIGN
- **Ground acceleration** : $a = 0.25$ g
- **Behaviour factor** : 6

![Diagram of the building and structural data](image-url)
I - GENERAL INFORMATION

The structural model of the frame is given in fig.1. The geometry of the structure and the position of the columns was decided taking into account the rooms layout of these residential buildings. In fig.2 is shown the typical arrangement of the single units.

EC8 - part 1.3 It is important to observe that, for "q = 6", all the frame members must have clause 3.5.2.1 class 1 cross sections. In this calculation example, the profiles that have been adopted fulfil this indirect structural ductility requirement.

The described calculations refer to one of the nine transversal frames of the building; in this way we refer to a simplified two dimensional model of the structure. The joints between the beams and the columns were supposed to be full strength connections and the loads that have been considered refer to a portion of building whose width is equal to the distance between the frames (thus, in this case, we take into account a portion of 5 m).

To facilitate the comprehension of the calculations, at the beginning of each section is reproduced a copy of the general flow chart "FC1" in which is evidenced the design phase that is currently being carried out.

II - LOADS

The following loads have been considered:

EC1 - part 2.1 Chapter 2 A) Self weight loads (SWL)

These include the weight of the steel frame members only and are calculated automatically by the codes used in this calculation example ("Stranger" and "PEP micro").
To take into account the weight of the joint elements (i.e. end plates, stiffeners, welds, bolts) these elementary self weight loads have been increased by 10% in the analysis and thus a $\lambda = 1.1$ load factor is always adopted.

**B) Dead loads (DL)**

**Levels I-II**

- r.c. slab 3 KN/m²
- secondary steel members self weight 0.5 KN/m²
- partition walls and ceiling covering 0.5 KN/m²
- pavement elements 0.5 KN/m²

Total uniform loads 4.5 KN/m²

Total uniform load for a 5 m wide building portion 5 x 4.5 = 22.5 KN/m

**C) Imposed loads (IL)**

**Levels I** (and also II if these actions are greater than the snow loads)

clause 4.5.2 category A 2 KN/m²

Total uniform load for 5 m building portion 2 x 5 = 10 KN/m

**D) Snow loads (SL)**

**Level II**

The snow loads are given by:

\[ S = \mu_i \cdot C_e \cdot C_t \cdot S_k \]

In our case:

clause 5.7.2 $\mu_i = 0.8$ (monopitch roof with $\alpha \geq 0$)

$C_e = 1$

$C_t = 1$

annex A - A10 $S_k = 0.9$ KN/m² (Genova, Italy, A=0)
\[ S = 0.8 \cdot 1 \cdot 0.9 = 0.72 \text{ KN/m}^2 \]

It is important to add that in this case the imposed loads have been considered for the II storey, since these have a higher value than the snow loads.

**E) Wind loads (WL)**

The pressure acting on the external surfaces is:

\[ w_e(z) = c_p \cdot c_{size} \cdot c_e(z) \cdot q_{ref} \]

in which:

\[ c_{size} = 0.49 \left[ 1 + 0.063 \cdot \ln \frac{3300}{l} \right]^2 \]

\[ c_e(z) = c_r(z)^2 \cdot c_t(z)^2 \cdot \left[ 1 + \frac{1.4}{c_r(z) \cdot c_t(z)} \right] \]

in which:

\[ c_r(z) = c_r(z_{min}) \quad \text{for } 0 < z \leq z_{min} \]

\[ C_t = 1 \quad \text{(for flat terrains)} \]

\[ q_{ref} = \frac{\rho \cdot v_{ref}^2}{2} \]

in which:

\[ v_{ref} = v_{ref,0} \cdot (1 + K_a \cdot a_s) \cdot c_{sem} \cdot c_{dir} \]

**In our case, the input parameters are:**

\[ c_p \]

annex B - 1.2

\[ \begin{align*}
  & c_p = +0.8 \quad \text{Upwind face} \\
  & = -0.3 \quad \text{Downwind face}
\end{align*} \]

\[ l = \sqrt{7^2 + 40^2} = 40.6 \text{ m} \]

annex A - 4

terrain roughness: type IV
\( \text{K}_r = 0.24 \)
\( z_0 = 1 \text{ m} \)
\( z_{\text{min}} = 16 \text{ m} \)

annex A - 9.11
\( \rho = 1.25 \text{ Kg/m}^3 \)
\( v_{\text{ref,0}} = 28 \text{ m/s} \) (for Genova, Italy, cat.3)
\( K_a = 0.55 \)
\( a_s = 0 \)
\( C_{\text{lem}} = 1 \)
\( C_{\text{dir}} = 1 \)

The wind pressure has been represented by a set of storey loads. These forces have been calculated with the Lotus 1-2-3 spreadsheet given in table 1, that performs a simple execution of the above written formulas. These actions were subsequently applied to each level taking into account that +0.8 corresponds to the upwind face and -0.3 corresponds to the downwind face of the buildings.

**EC3 - 5.2.4.3**

**F) Frame imperfections loads (FIL)**

**Levels I-II**

The initial frame imperfections may be represented by a set of horizontal forces that are given, for the generic storey (i), by:

\[ F_i = \phi \cdot W_i \]

where:

\[ \phi = \frac{k_s \cdot k_c}{200} \]

**TOTAL VERTICAL STOREY LOADS**

**INITIAL SWAY IMPERFECTION**

\[ k_c = \sqrt{0.5 + \frac{1}{n_c}} \quad \text{with } k_s \leq 1 \]

\[ k_s = \sqrt{0.2 + \frac{1}{n_s}} \quad \text{with } k_s \leq 1 \]

in which:

\( n_c = \text{number of columns} \)

\( n_s = \text{number of storeys} \)
In our case we have:

\[ n_c = 3 \quad k_c = 0.913 \]
\[ n_b = 2 \quad k_b = 0.837 \]

\[ \phi = \frac{0.913 \cdot 0.837}{200} = 0.00382 \]

In the calculation of the total storey loads, it is necessary to sum to the vertical loads calculated in B) and C) the weight of the frame beams and columns; in our case we have supposed 0.5 KN/m².

For both storeys we obtain:

\[ W = (4.5 + 2 + 0.5) \times 11.5 \times 5 = 402.5 \text{ KN} \]

The storey equivalent imperfection loads can now be calculated:

\[ F = 0.00382 \cdot 402.5 = 1.54 \text{ KN} \]

These horizontal loads are applied half in one side of the frame, the other half on the other.

**G) Earthquake loads (EL)**

The following input data is considered:

- Frame natural period
  \[ T_0 = 0.8 \text{ s} \]
- Structural viscous damping ratio
  \[ \zeta = 5\% \]
- Behaviour factor
  \[ q = 6 \]
- Ground acceleration
  \[ \frac{a}{g} = 0.25 \]
- Subsoil class
  type B

When performing a seismic analysis, if the following condition regarding the natural vibration period is fulfilled:
it is possible to represent the seismic actions with a set of storey horizontal loads, performing a simplified dynamic analysis. These forces, for generic storey \( i \), are given by:

\[
F_i = \beta(T_0) \cdot z_i \cdot \frac{\sum W_j}{\sum W_j \cdot z_j}
\]

in which:

\[
\beta(T_0) = \frac{a}{g} \cdot s \cdot \left[ 1 + \frac{T_0}{T_1} \left( \frac{\eta \cdot \beta_0}{q} - 1 \right) \right] \quad 0 < T_0 < T_1
\]

\[
\beta(T_0) = \frac{a}{g} \cdot s \cdot \left( \frac{T_0}{T_2} \right)^k \quad T_2 < T_0
\]

In any case:

\[
\beta(T_0) \geq 0.20 \cdot \left( \frac{a}{g} \right)
\]

\[
W_j = W_{p,j} + W_{v,j} \cdot \psi_{2i} \cdot \phi_j
\]

in which:

\( W_{p,j} = \) storey(j) permanent loads

\( W_{v,j} = \) storey(j) variable loads

\( \phi_j = \) storey(j) load factor

In our case:

\( 0.8 < 2 \times 0.6 = 1.2 \)

it is possible to perform a simplified dynamic analysis

\( \phi_j = \)

\begin{align*}
0.5 & \quad \text{First storey} \\
1 & \quad \text{Second storey} \\
\eta = 1
\end{align*}
These forces have been calculated with the Lotus 1-2-3 spreadsheet given in table 2, that performs a simple execution of the above written formulas. These actions, divided by two, were then applied to the two end nodes of each level.

III - LOAD CASES

The following load combination rules have been used:

EC1 - part 1 - For load cases **without seismic loads**

clause 9.4.5  
**A) Ultimate limit states (ULS)**

\[ \sum_j \gamma_{G_j} \cdot G_{k,j} + 1.35 \cdot \sum_i Q_{k,i} \]

\[ \sum_j G_{k,j} + 0.9 \cdot \sum_i Q_{k,i} \]

clause 9.5.5  
**B) Serviceability limit states (SLS)**

- For load cases **with seismic loads**

EC8 - part1.1  
**A) Ultimate limit states (ULS)**

clause 6.2

\[ \gamma \cdot E + \sum_j G_{k,j} + \sum_i \Psi_{2,j} \cdot Q_{k,i} \]

where:
G_{k,j} = \text{characteristic value of the permanent actions}
Q_{k,i} = \text{characteristic value of the variable actions}

\gamma_{Gj} = \text{partial factors for permanent actions}
\gamma = \text{seismic actions importance factor (=1 in our case)}
\Psi_{2i} = \text{load combination coefficient}

**B) Serviceability limit states (SLS)**

The same combination rule used for the ULS was used also for serviceability checks; as it will be more clear further on, appropriate factors are introduced by the EC8 to differentiate the entity of the actions for the two limit states.

The loads considered in this calculation example are:

1) **Permanent actions**

   - SWL
   - DL
   - FIL

2) **Variable actions**

   - IL
   - SL
   - WL

3) **Accidental actions**

   - EL

It is important to notice that the equivalent frame imperfections loads have been considered as "permanent" actions since these characterise the geometrical imperfections of the frame elements and thus are a permanent characteristic of the structure.

In addition, since the frame geometry is asymmetrical, it was necessary to consider in the cases the horizontal loads (WL, FIL, and EL) acting in both directions. The direction of these actions is indicated in every single case by adding "L" or "R" to the load case name.

It is also important to remember that the self weight loads (SWL) have already been increased by 10%.

The **ULS load cases** that have been considered in this calculation example are:
1) \(0.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + 1.35 \cdot FIL_R\)
2) \(0.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + 1.35 \cdot FIL_R + 1.35 \cdot WL_R\)
3) \(0.00 \cdot SWL + 1.00 \cdot DL + 0.3 \cdot IL + 1.00 \cdot FIL_R + 1.00 \cdot EL_R\)
4) \(0.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + 1.35 \cdot FIL_L\)
5) \(0.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + 1.35 \cdot FIL_L + 1.35 \cdot WL_L\)
6) \(0.00 \cdot SWL + 1.00 \cdot DL + 0.3 \cdot IL + 1.00 \cdot FIL_L + 1.00 \cdot EL_L\)

The SLS load cases that have been considered are:
7) \(0.00 \cdot SWL + 1.00 \cdot DL + 0.9 \cdot IL + 1.00 \cdot FIL_R + 0.9 \cdot WL_R\)
8) \(0.00 \cdot SWL + 1.00 \cdot DL + 0.9 \cdot IL + 1.00 \cdot FIL_L + 0.9 \cdot WL_L\)

IV - PRELIMINARY SIZING OF MEMBERS

The pre dimensioning of the frame beams and columns was performed with simple preliminary calculations. The chosen profiles are shown in fig.3

V - SLS CHECKS
A preliminary first order analysis of the frame has been carried out. The numbering of nodes and members is given in fig. 4. The SLS verifications that have been performed regard horizontal displacement checks of the different storeys of the frame.

The calculations methodology and the used formulas differ from each other for load cases with seismic actions to those without seismic actions.

A) Load cases with seismic loads

According the Eurocode 8, the horizontal deflection limits for the relative storey displacements "\(d_{r,i}\)" are:

\[
d_{r,i} \leq \nu \cdot 0.006 \cdot h
\]

where:
\(\nu\) = reduction factor that takes into account the difference in the loads entity between an "ordinary" (serviceability) earthquake and an "exceptional" (ultimate) one.

\(h\) = storey height

This displacement can be calculated, for storey(i), by:

\[
d_{r,i} = q \cdot d_{st,i} + d_{mr,i}
\]

where:
\(q\) = behaviour factor
\( d_{\text{sys},i} \) = design horizontal displacement (for storey(i)), due to the seismic forces, obtained from a first order analysis

\( d_{\text{res},i} \) = design horizontal displacement (for storey(i)), due to the other (horizontal and vertical) loads, obtained from a first order analysis

These horizontal storey displacements have been obtained calculating the mean value between the deflections of the two extreme nodes of each storey.

\text{clause 4.2.1} \quad \text{In our case, for } v = 2.5

\[ q = 6 \]

\text{for load case 3):}

- for storey I:
  \[ d_{r,1} = 6 \cdot \frac{0.918 + 0.919}{2} + \frac{0.0424 + 0.0620}{2} = 5.563 < 0.006 \cdot 2.5 \cdot 380 = 5.70 \text{ cm} \]

- for storey II:
  \[ d_{r,II} = 6 \cdot \frac{0.392 + 0.391}{2} + \frac{0.0440 + 0.0014}{2} = 2.372 < 0.006 \cdot 2.5 \cdot 320 = 4.80 \text{ cm} \]

\text{for load case 6):}

- for storey I:
  \[ d_{r,1} = 6 \cdot \frac{0.918 + 0.919}{2} + \frac{0.0584 + 0.0390}{2} = 5.560 < 0.006 \cdot 2.5 \cdot 380 = 5.70 \text{ cm} \]

- for storey II:
  \[ d_{r,II} = 6 \cdot \frac{0.392 + 0.391}{2} + \frac{-0.0042 + 0.0383}{2} = 2.366 < 0.006 \cdot 2.5 \cdot 320 = 4.80 \text{ cm} \]

\text{B) Load cases without seismic loads}

According to the EC3, the horizontal deflection limits for multi-storey buildings are:

\text{EC3} \quad \text{- in each storey:}

\[ d_i \leq \frac{h_i}{300} \]

\text{clause 4.2.2 (4)}

\text{- for the structure as a whole:}

\[ d_{\text{tot}} \leq \frac{h}{500} \]
where:
\[
\begin{align*}
d_{\text{tot}} & = \text{total frame horizontal deflection} \\
d_i & = \text{storey(i) relative horizontal deflection} \\
h & = \text{frame total height} \\
h_i & = \text{storey(i) height}
\end{align*}
\]

The storeys horizontal deflections were obtained, as it has been done for the cases with seismic loads, by calculating the average value between the horizontal displacements of the two end nodes of each storey. It is also necessary to add that the verification of the storey relative horizontal displacement has only been performed in the first storey since it is less stiff than the second level and thus enables greater displacements.

In our case:

for load case 7):

- for storey 1:
\[
d_i = 0.265 \leq \frac{h_i}{300} = \frac{380}{300} = 1.27\text{cm}
\]

- for the structure as a whole:
\[
d_{\text{tot}} = 0.364 \leq \frac{h}{500} = \frac{700}{500} = 1.40\text{cm}
\]

for load case 8):

- for storey 1:
\[
d_i = 0.261 \leq \frac{h_i}{300} = \frac{380}{300} = 1.27\text{cm}
\]

- for the structure as a whole:
\[
d_{\text{tot}} = 0.353 \leq \frac{h}{500} = \frac{700}{500} = 1.40\text{cm}
\]

VI - ULS ELASTIC CALCULATION OF THE INTERNAL FORCES

section 5.2

section 5.2.5.2 A) CHECK FOR "SWAY-NON SWAY" BEHAVIOUR
Before the internal forces and moments are calculated and used in member and cross-sectional checks, it is necessary to check if the non-linear geometrical effects (second order) have to be taken into account or not.

A frame is classified as "sway" for a given load case if, for at least one of the storeys, the following condition is satisfied:

\[
\left( \frac{\delta_i}{h_i} \right) \left( \frac{\sum V_i}{\sum H_i} \right) > 0.1
\]

where:
- \( h_i \) = storey(i) height
- \( \delta_i \) = relative horizontal displacement of storey(i)
- \( \sum V_i \) = total vertical reaction at bottom of storey(i) from first order analysis
- \( \sum H_i \) = total horizontal reaction at the bottom of storey(i) from first order analysis

The relative storey displacements have been obtained with different methods:

1) Load cases with seismic loads

These are calculated in the same way as for the SLS checks:

\[
d_{r,i} = q \cdot d_{\text{str},i} + d_{\text{res},i}
\]

2) Load cases without seismic loads

These are taken directly from the first order analysis of the frame.
The values of the "sway-non sway" ratio for the ULS load cases (1, 2, 3, 4, 5 and 6) of this specific example are given in the Lotus 1-2-3 spreadsheets given in tables 3, 4 and 5.

The frame results to be "sway" for load cases 3 and 6; in these load cases it is necessary to take into account the non linear geometrical effects performing a second order analysis. For load cases 1, 2, 4 and 5 instead, it is sufficient to take directly the first order internal forces and moments. It is interesting to observe that these are all the non seismic load cases; it is thus evident that being the entity of the seismic actions much greater than that of the other loads, the frame elements are dimensioned mainly on the earthquake effects and the structure results to be over dimensioned for the other actions.

clause 5.2.4.2(4) B) CHECK FOR MEMBER IMPERFECTIONS

According to the EC3, for load cases in which the frame results to be "sway", it is necessary to check if the effects of member imperfections can be neglected or not.

For this purpose, if in a compression member with moment resisting connections the given formula is not fulfilled then it is necessary to take into account these imperfections in the global analysis.

\[
\overline{\lambda} \leq 0.5 \cdot \sqrt{\frac{A \cdot f_y}{N_{sd}}}
\]

where:
In the scope of this research project, the above written condition should be checked for all the columns of the frame, taking the internal forces and moments of load cases 3 and 6. As an example, we can take element 5 (HEB 260) with the internal forces of case 3.

In our case:

\[ A = 118 \text{ cm}^2 \]
\[ N_{sd} = 151 \text{ KN} \]
\[ f_y = 355 \text{ N/mm}^2 \]

\[ \lambda = \frac{\lambda}{93.9 \cdot \varepsilon} \cdot \sqrt{\beta_4} = \frac{380/11.2}{93.9 \cdot 0.814} \cdot \sqrt{1} = 0.444 \]

and thus:

\[ \lambda = 0.444 < 0.5 \cdot \sqrt{\frac{118 \cdot 355 \cdot 10^{-1}}{151}} = 2.634 \]

It is not necessary to account for the member imperfections in the global analysis.

C) SECOND ORDER ANALYSIS

The EC3 enables to adopt the rigorous second order calculation method together with other indirect second order analysis methods.

1) Rigorous second order analysis method
The direct second order analysis has been performed with "PEP micro". The internal forces and moments determined in the calculation can be used directly for the local member and cross sectional checks that are performed using the "non sway" buckling lengths.

5.2.6.2 (8)  

2) Sway mode buckling length method

This is one of the indirect second order analysis methods indicated by the EC3; for column member and cross-sectional checks we take directly the first order analysis results and adopt "sway mode" buckling lengths; for the beams it is necessary to increase by 20% the "sway moments"; the beams local member and cross-sectional checks are then performed with the increased values of these moments.
This is an indirect second order calculation method that is applicable only if, for a given load case, all the storeys of the frame satisfy the following equation:

\[
\left( \frac{\delta_i}{h_i} \right) \left( \frac{\sum V_i}{\sum H_i} \right) \leq 0.25
\]

The "sway moments" that result from the first order analysis have to be increased to take into account the additional effects due to the "sway" behaviour of the frame.

For each different storey the value of load factor \( \alpha_i \) is obtained with the formula:

\[
\alpha_i = \frac{1}{1 - \left( \frac{\delta_i}{h_i} \right) \left( \frac{\sum V_i}{\sum H_i} \right)}
\]

In our case:

- for load case 3)

LEVEL I

\[
\alpha_i = \frac{1}{1 - 0.1621} = 1.194
\]
LEVEL II

\[ \alpha_u = \frac{1}{1 - 0.0628} = 1.068 \]

- for load case 6)

LEVEL I

\[ \alpha_i = \frac{1}{1 - 0.1621} = 1.193 \]

LEVEL II

\[ \alpha_u = \frac{1}{1 - 0.0626} = 1.067 \]

According to the EC3, these amplification factors should be used to multiply the "sway moments" (that are those that result from the horizontal loads and also the vertical loads if either the structure or the loading is asymmetrical). This methodology, as it has been evidenced at point 2.2.7 of the report, shows to be not applicable in the case of asymmetrical structure since it not very clear if all the moments that derive from the vertical loading have to be multiplied by the relevant values of "\( \alpha_i \)" or only part of these forces.

The alternative method that has been chosen for this specific calculation example (the loads that give the horizontal translation of the storeys are multiplied by the relevant values of "\( \alpha_i \)"") is based on the same principle as the one given in the Eurocode; the methodology evidences that the values of the internal forces and moments obtained are conservative to the point that the frame beams and columns sections have to be incremented.

Nevertheless, if this method is adopted it is necessary to multiply all the loads (horizontal and vertical) since the frame is asymmetrical.

As a consequence, for each different storey of the frame we obtain the following load cases with increased partial load factors:

- load case 3)

LEVEL I

\[(1.194 \cdot 1.00) \cdot SWL + (1.194 \cdot 1.00) \cdot DL + (1.194 \cdot 0.3) \cdot IL + (1.194 \cdot 1.00) \cdot FIL_R + (1.194 \cdot 1.00) \cdot EL\]

LEVEL II
With these new load factors for load cases 3) and 6), a new first order analysis was performed. The results are used directly in the local member and cross-sectional verifications in which we adopt the "non sway mode" buckling lengths.

In tables 6 and 7 it is possible to compare the internal forces of:
- Rigorous second order method
- Moment amplification method (with "αi" applied to the loads that give the "sway moments").
- First order method for COLUMNS

- Rigorous second order method
- Moment amplification method (with "αi" applied to the loads that give the "sway moments").
- Sway mode buckling lengths method
- First order method for BEAMS

The comparison between the two indirect second order methods for the columns is only possible comparing the utilisation factors (expressed as a percentage) that are obtained from the bending-axial force buckling member checks.
The following remarks can be made:

*columns* - if we examine the values of the bending moments, it is visible that those that are calculated with a rigorous II order analysis are similar to those of the I order calculation. In addition, the "moment amplification" calculation gives values for the moments that are much higher than those that derive from the II order analysis, and thus the method is very much conservative.
Civil building (CIV) analysis
Frame structural model

Note: the dimensions are indicated in millimetres

ECSC Project 7210-SA/419
Practical application of EUROCODE 3
to multi-storey buildings with steel
"Sway Frame" structures
Civil building (CIV) analysis
Building units arrangement

Note: the dimensions are indicated in metres

ECSC Project 7210-SA/419
Practical application of EUROCODE 3
to multi-storey buildings with steel "Sway Frame" structures
### Civil building (CIV) seismic analysis

**Member profiles**

<table>
<thead>
<tr>
<th>Member</th>
<th>Profile</th>
</tr>
</thead>
<tbody>
<tr>
<td>ftpur.</td>
<td>IPE 400</td>
</tr>
</tbody>
</table>

**IPE 400**

<table>
<thead>
<tr>
<th>CN</th>
<th>CD</th>
</tr>
</thead>
<tbody>
<tr>
<td>LU</td>
<td>LU</td>
</tr>
</tbody>
</table>

**X**

### Practical application of EUROCODE 3 to multi-storey buildings with steel "Sway Frame" structures

ECSC Project 7210-SA/419
FIGURE 4

Node numbers

Element numbers
### Table 1

<table>
<thead>
<tr>
<th>Level</th>
<th>Height building portion (m)</th>
<th>Reference elevation (m)</th>
<th>Positive pressure coefficient</th>
<th>Negative pressure coefficient</th>
<th>Positive storey load (KN)</th>
<th>Negative storey load (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>3.5</td>
<td>3.8</td>
<td>0.8</td>
<td>0.3</td>
<td>7.53</td>
<td>2.83</td>
</tr>
<tr>
<td>II</td>
<td>1.6</td>
<td>7</td>
<td>0.8</td>
<td>0.3</td>
<td>3.44</td>
<td>1.29</td>
</tr>
<tr>
<td>III</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>IV</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>V</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>VI</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>VII</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>VIII</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>VIII</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>IX</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>
**Title:** Civil building (CIV) calculation example

**Date:**

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>Wpi</th>
<th>Wvi</th>
<th>fi_i</th>
<th>Wtot_i</th>
<th>Zi</th>
<th>Fi</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>287.5</td>
<td>115</td>
<td>0.5</td>
<td>304.75</td>
<td>3.8</td>
<td>18.29</td>
</tr>
<tr>
<td>II</td>
<td>287.5</td>
<td>115</td>
<td>1</td>
<td>322</td>
<td>7</td>
<td>35.60</td>
</tr>
<tr>
<td>III</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.00</td>
</tr>
<tr>
<td>IV</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.00</td>
</tr>
<tr>
<td>V</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.00</td>
</tr>
<tr>
<td>VI</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.00</td>
</tr>
<tr>
<td>VII</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.00</td>
</tr>
<tr>
<td>VIII</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**Table 2**

**Notes:**
- Wpi: dead loads of storey (i)
- Wvi: imposed loads of storey (i)
- Wtot_i: total storey (i) loads (with reduced imposed loads)
- Fi: storey (i) seismic loads
- Zi: storey (i) level height
- fi_i: storey (i) reduction load factor
### TABLE 3

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
<td>291</td>
<td>-9.350</td>
<td>0.000</td>
<td>2</td>
<td>0.055</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>558</td>
<td>-7.120</td>
<td>0.000</td>
<td>5</td>
<td>0.088</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>220</td>
<td>-6.390</td>
<td>0.000</td>
<td>8</td>
<td>0.088</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Storey total forces (KN) 1069 -4.160

Storey height 380

Verifica sway: \( \frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}}} \) = 0.071

### Storey 2

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1</td>
<td>2</td>
<td>144</td>
<td>-68.400</td>
<td>0.055</td>
<td>3</td>
<td>0.123</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>282</td>
<td>-32.900</td>
<td>0.088</td>
<td>6</td>
<td>0.083</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>107</td>
<td>-37.600</td>
<td>0.088</td>
<td>9</td>
<td>-0.005</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Storey total forces (KN) 533 -2.100

Storey height 320

Verifica sway: \( \frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}}} \) = 0.032

### Storey 1

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>1</td>
<td>282</td>
<td>-3.930</td>
<td>0.000</td>
<td>2</td>
<td>0.373</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>560</td>
<td>-19.200</td>
<td>0.000</td>
<td>5</td>
<td>0.405</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>227</td>
<td>-9.250</td>
<td>0.000</td>
<td>8</td>
<td>0.405</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Storey total forces (KN) 533 -8.500

Storey height 320

Verifica sway: \( \frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}}} \) = 0.146

### Storey 2

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>2</td>
<td>2</td>
<td>141</td>
<td>-65.600</td>
<td>0.373</td>
<td>3</td>
<td>0.555</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>282</td>
<td>-34.200</td>
<td>0.000</td>
<td>6</td>
<td>0.515</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>110</td>
<td>-39.900</td>
<td>0.405</td>
<td>9</td>
<td>0.110</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Storey total forces (KN) 533 -5.700

Storey height 380

Verifica sway: \( \frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}}} \) = 0.1621
### Table 4

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>3</td>
<td>2</td>
<td>74</td>
<td>26.900</td>
<td>5.550</td>
<td>3</td>
<td>7.946</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>168</td>
<td>32.400</td>
<td>0.000</td>
<td>6</td>
<td>7.923</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>72</td>
<td>-31.600</td>
<td>5.376</td>
<td>9</td>
<td>2.347</td>
</tr>
</tbody>
</table>

Storey total forces (KN) | 314 | -37.100 |

**Veritica sway:** \( \frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}} \times 10^6} \) = **0.0628**

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td>1</td>
<td>295</td>
<td>11.600</td>
<td>0.000</td>
<td>2</td>
<td>0.082</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>557</td>
<td>-2.190</td>
<td>0.000</td>
<td>5</td>
<td>0.048</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>216</td>
<td>-5.210</td>
<td>0.000</td>
<td>8</td>
<td>0.048</td>
</tr>
</tbody>
</table>

Storey total forces (KN) | 1068 | 4.200 |

**Veritica sway:** \( \frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}} \times 10^6} \) = **0.0435**

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>4</td>
<td>2</td>
<td>145</td>
<td>70.000</td>
<td>0.082</td>
<td>3</td>
<td>0.067</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>281</td>
<td>-31.600</td>
<td>0.048</td>
<td>6</td>
<td>0.107</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>106</td>
<td>-36.400</td>
<td>0.048</td>
<td>9</td>
<td>0.059</td>
</tr>
</tbody>
</table>

Storey total forces (KN) | 532 | 2.000 |

**Veritica sway:** \( \frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}} \times 10^6} \) = **0.0184**

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>1</td>
<td>305</td>
<td>16.900</td>
<td>0.000</td>
<td>2</td>
<td>0.399</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>556</td>
<td>-9.950</td>
<td>0.000</td>
<td>5</td>
<td>0.369</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>209</td>
<td>-2.310</td>
<td>0.000</td>
<td>8</td>
<td>0.369</td>
</tr>
</tbody>
</table>

Storey total forces (KN) | 1070 | 24.540 |

**Veritica sway:** \( \frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}} \times 10^6} \) = **0.0441**

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>5</td>
<td>2</td>
<td>148</td>
<td>72.900</td>
<td>0.399</td>
<td>3</td>
<td>0.499</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>281</td>
<td>-30.300</td>
<td>0.369</td>
<td>6</td>
<td>0.170</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>104</td>
<td>-34.100</td>
<td>0.369</td>
<td>9</td>
<td>0.539</td>
</tr>
</tbody>
</table>

Storey total forces (KN) | 533 | 8.500 |

**Veritica sway:** \( \frac{N_{\text{tot}} \times \text{st.disp.}}{H \times V_{\text{tot}} \times 10^6} \) = **0.0223**
### Eurocode 3: Sway - Non Sway Criteria

**Title:** Civil building (CIV) - Seismic design - hinged column bases

#### Storey Load Combinations

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6</td>
<td>1 205</td>
<td>21.200</td>
<td>0.000</td>
<td>2 5.566 5.566</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 323</td>
<td>31.000</td>
<td>0.000</td>
<td>5 5.553 5.553</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7 103</td>
<td>4.750</td>
<td>0.000</td>
<td>8 5.553 5.553</td>
</tr>
</tbody>
</table>

**Storey total forces (KN):** 631 56.950

**Storey height:** 380  mean rel. disp. (cm) 5.560

**Verifica sway:** \[
\left( N_{\text{tot}} \times \text{st.disp.} \right) / \left( H \times V_{\text{tot}} \right) = 0.1621
\]

#### Storey Load Combinations

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>6</td>
<td>2 96</td>
<td>54.100</td>
<td>5.566</td>
<td>3 7.914 2.348</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5 164</td>
<td>5.370</td>
<td>5.553</td>
<td>6 7.937 2.384</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8 54</td>
<td>-11.600</td>
<td>5.553</td>
<td>9 7.937 2.384</td>
</tr>
</tbody>
</table>

**Storey total forces (KN):** 314 37.130

**Storey height:** 320  mean rel. disp. (cm) 2.366

**Verifica sway:** \[
\left( N_{\text{tot}} \times \text{st.disp.} \right) / \left( H \times V_{\text{tot}} \right) = 0.0626
\]

#### Storey Load Combinations

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2 94</td>
<td>54.100</td>
<td>5.566</td>
<td>3 7.914 2.348</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5 163</td>
<td>5.370</td>
<td>5.553</td>
<td>6 7.937 2.384</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8 54</td>
<td>-11.600</td>
<td>5.553</td>
<td>9 7.937 2.384</td>
</tr>
</tbody>
</table>

**Storey total forces (KN):** 0 0.000

**Storey height:** mean rel. disp. (cm) ERR

**Verifica sway:** \[
\left( N_{\text{tot}} \times \text{st.disp.} \right) / \left( H \times V_{\text{tot}} \right) = ERR
\]

#### Storey Load Combinations

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes total disp. relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2 94</td>
<td>54.100</td>
<td>5.566</td>
<td>3 7.914 2.348</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5 163</td>
<td>5.370</td>
<td>5.553</td>
<td>6 7.937 2.384</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8 54</td>
<td>-11.600</td>
<td>5.553</td>
<td>9 7.937 2.384</td>
</tr>
</tbody>
</table>

**Storey total forces (KN):** 0 0.000

**Storey height:** mean rel. disp. (cm) ERR

**Verifica sway:** \[
\left( N_{\text{tot}} \times \text{st.disp.} \right) / \left( H \times V_{\text{tot}} \right) = ERR
\]

**TABLE 5**
## Internal Forces and Moments

### Table 6

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>N [kN]</td>
<td>V [kN]</td>
<td>M [kN*m]</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>6</td>
<td>200</td>
<td>-21.2</td>
<td>80.6</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>6</td>
<td>92.1</td>
<td>-54.1</td>
<td>107</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>5</td>
<td>6</td>
<td>316</td>
<td>-31.0</td>
<td>118</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>6</td>
<td>6</td>
<td>158</td>
<td>5.38</td>
<td>25.2</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>8</td>
<td>6</td>
<td>99.0</td>
<td>-4.75</td>
<td>18.1</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>9</td>
<td>6</td>
<td>50.7</td>
<td>11.6</td>
<td>-15.7</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>N [kN]</th>
<th>V [kN]</th>
<th>M [kN*m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2nd Order</td>
<td>201</td>
<td>-21.1</td>
<td>82.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>2nd Order</th>
<th>N [kN]</th>
<th>V [kN]</th>
<th>M [kN*m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2nd Order</td>
<td>201</td>
<td>-21.1</td>
<td>82.1</td>
</tr>
</tbody>
</table>

**EC3: "SWAY FRAMES" Analysis Results**

**Title:** Civil building (CIV) - Seismic design - COLUMNS

**Date:**

**INTERNAL FORCES AND MOMENTS**

**N (kN)**

<table>
<thead>
<tr>
<th>1st Order</th>
<th>1st Order - Amplified Moments</th>
<th>1st Order - Sway Mode Buck. Lengths</th>
<th>2nd Order</th>
</tr>
</thead>
<tbody>
<tr>
<td>225</td>
<td>225</td>
<td></td>
<td></td>
</tr>
<tr>
<td>98.6</td>
<td>98.6</td>
<td>116</td>
<td></td>
</tr>
<tr>
<td>316</td>
<td>316</td>
<td>129</td>
<td></td>
</tr>
<tr>
<td>158</td>
<td>158</td>
<td>28.1</td>
<td></td>
</tr>
<tr>
<td>113</td>
<td>113</td>
<td>18.4</td>
<td></td>
</tr>
<tr>
<td>54.0</td>
<td>54.0</td>
<td>13.3</td>
<td></td>
</tr>
</tbody>
</table>

**V (kN)**

<table>
<thead>
<tr>
<th>1st Order</th>
<th>1st Order - Amplified Moments</th>
<th>1st Order - Sway Mode Buck. Lengths</th>
<th>2nd Order</th>
</tr>
</thead>
<tbody>
<tr>
<td>-21.2</td>
<td>-21.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-59.7</td>
<td>-59.7</td>
<td>116</td>
<td></td>
</tr>
<tr>
<td>-34.0</td>
<td>-34.0</td>
<td>129</td>
<td></td>
</tr>
<tr>
<td>6.78</td>
<td>6.78</td>
<td>28.1</td>
<td></td>
</tr>
<tr>
<td>-4.84</td>
<td>-4.84</td>
<td>18.4</td>
<td></td>
</tr>
<tr>
<td>13.3</td>
<td>13.3</td>
<td>16.8</td>
<td></td>
</tr>
</tbody>
</table>

**M (kN*m)**

<table>
<thead>
<tr>
<th>1st Order</th>
<th>1st Order - Amplified Moments</th>
<th>1st Order - Sway Mode Buck. Lengths</th>
<th>2nd Order</th>
</tr>
</thead>
<tbody>
<tr>
<td>80.6</td>
<td>80.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>107</td>
<td>107</td>
<td></td>
<td></td>
</tr>
<tr>
<td>118</td>
<td>118</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25.2</td>
<td>25.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>28.1</td>
<td>28.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>18.4</td>
<td>18.4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**ECSC Project SA/419 - "EC3 Sway Frames"**

**Date:**

- **Elem. Num.**
- **Node Num.**
- **Comb. Num.**
- **N (kN)**
- **V (kN)**
- **M (kN*m)**

**Table 6**
<table>
<thead>
<tr>
<th>Num.</th>
<th>Node</th>
<th>Comb.</th>
<th>N (kN)</th>
<th>V (kN)</th>
<th>M (kN*m)</th>
<th>N (kN)</th>
<th>V (kN)</th>
<th>M (kN*m)</th>
<th>N (kN)</th>
<th>V (kN)</th>
<th>M (kN*m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>2</td>
<td>6</td>
<td>103</td>
<td>-147</td>
<td></td>
<td>121</td>
<td>-168</td>
<td></td>
<td>124</td>
<td>-176</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>5</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>-67.0</td>
<td>85.2</td>
<td></td>
<td>-45.0</td>
<td>92.1</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>5</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>85.2</td>
<td>-104</td>
<td></td>
<td>100</td>
<td>-120</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>-45.0</td>
<td>-3.29</td>
<td></td>
<td>-55.1</td>
<td>-7.20</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>3</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>92.1</td>
<td>-107</td>
<td></td>
<td>98.6</td>
<td>-116</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>6</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>-78.4</td>
<td>-62.6</td>
<td></td>
<td>-83.2</td>
<td>-65.7</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>6</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>79.5</td>
<td>-87.8</td>
<td></td>
<td>84.8</td>
<td>-93.7</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>9</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>-50.7</td>
<td>-15.7</td>
<td></td>
<td>-54.0</td>
<td>-16.8</td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 7**
MULTI-STOREY PARKING BUILDING (PAR)

NON SEISMIC CALCULATION
Type of building : Multi-storey parking (PAR)
Type of design : Non seismic

BUILDING AND STRUCTURAL DATA

- Dimensions
  Length (m) : 60
  Width (m) : 32
  Height (m) : 16.1

- Storeys
  Number : 5

- Structural system
  transversal direction: the bracing system for the horizontal loads is given by nine beam-column rigid jointed frames whose distance is 7.5 m. The columns of these structures are supposed to be hinged at the bases.
  longitudinal direction: the bracing system is obtained with a number of cross bracing structures that are added to the frame that is supposed to have beam-column hinged joints.

MATERIAL

Steel Grade (for profiles, plates and welds) : Fe 510
Bolts grade : 8.8
I - GENERAL INFORMATION

The structural model of the frame is given in fig.1. The geometry of the structure and the position of the columns was decided taking into account the layout of the parking areas and the circulation ways that is shown, as a typical solution for all the storeys, in fig.2.

The described calculations refer to one of the nine transversal frames of the building; in this way we refer to a simplified two dimensional model of the structure. The joints between the beams and the columns were supposed to be full strength connections and the loads that have been considered refer to a portion of building whose width is equal to the distance between the frames (thus, in this case, we take into account a portion of 7.5 m).

To facilitate the comprehension of the calculations, at the beginning of each section is reproduced a copy of the general flow chart "FC1" in which is evidenced the design phase that is currently being carried out.

II - LOADS

The following loads have been taken into account:

A) Self weight loads (SWL)

These include the weight of the steel frame members only and are calculated automatically by the codes used in this calculation example ("Stranger" and "PEP micro").

To take into account the weight of the joint elements (i.e. end plates, stiffeners, welds, bolts) these elementary self weight loads have been increased by 10% in the analysis and thus a $\lambda = 1.1$ load factor is always adopted.
**B) Dead loads (DL)**

*Levels I-II-III-IV-V*

- r.c. slab 4 KN/m²
- secondary steel members self weight 0.5 KN/m²
- finishing coat 0.5 KN/m²

Total uniform loads 5 KN/m²

Total uniform load for 7.5 m wide building portion

\[7.5 \times 5 = 37.5\text{ KN/m}\]

---

**C) Imposed loads (IL)**

*Levels I-II-III-IV (and also V if these are greater than the snow loads)*

clause 4.6.2 category F

Total uniform load for 7.5 m building portion

\[2 \times 7.5 = 15\text{ KN/m}\]

---

**D) Snow loads (SL)**

*Level V*

The snow loads are given by:

\[S = \mu_i \cdot C_e \cdot C_t \cdot S_k\]

In our case:

clause 5.7.2 \(\mu_i = 0.8\) (monopitch roof with \(\alpha \neq 0\))

\(C_e = 1\)

\(C_t = 1\)

annex A - A10 \(S_k = 90\text{ KN/m}^2\) (Genova, Italy, \(A = 0\))

\[S = 0.8 \cdot 1 \cdot 1 \cdot 0.9 = 0.72\text{ KN/m}^2\]

It is important to add that for storey V imposed loads have been considered instead since these have a higher value than the snow loads.

---

**E) Wind loads (WL)**
The pressure acting on the external surfaces is:

\[ w_e(z) = c_p \cdot c_{size} \cdot c_e(z) \cdot q_{ref} \]

in which:

\[ c_{size} = 0.49 \cdot \left(1 + 0.063 \cdot \ln \frac{3300}{l}\right)^2 \]

**SIZE COEFFICIENT**

\[ c_e(z) = c_e(z) \cdot c_e(z) \left[1 + \frac{14}{c_e(z) \cdot c_e(z)}\right] \]

**EXPOSURE COEFFICIENT**

in which:

\[ c_e(z) = c_e(z_{min}) \quad \text{for } 0 < z < Z_{min} \]

\[ c_e(z) = K_r \cdot \ln \left(\frac{z}{z_o}\right) \quad \text{for } z > Z_{min} \]

**ROUGHNESS COEFFICIENT**

\[ C_t = 1 \quad \text{for flat terrains} \]

**TOPOGRAPHY COEFFICIENT**

\[ q_{ref} = \frac{P \cdot v_{ref}^2}{2} \]

**WIND PRESSURE**

in which:

\[ v_{ref} = v_{ref,0} \cdot (1 + K_a \cdot a_s) \cdot c_{tem} \cdot c_{dir} \]

\[ c_p \]

**PRESSURE COEFFICIENT**

In our case, the input parameters are:

**annex B - 1.2**

\[ c_p = +0.8 \quad \text{Upwind face} \]

\[ = -0.3 \quad \text{Downwind face} \]

\[ l = \text{diagonal size of loaded area} \]

\[ = \sqrt{16.1^2 + 60^2} = 62. \text{ m} \]

**annex A - 4**

terrain roughness: type IV

\[ K_r = 0.24 \]

\[ z_o = 1 \text{ m} \]

\[ Z_{min} = 16 \text{ m} \]

**annex A - 9.11**

\[ \rho = 1.25 \text{ Kg/m}^3 \]

\[ v_{ref,0} = 28 \text{ m/s} \quad \text{(for Genova, Italy, cat.3)} \]

\[ K_a = 0.55 \]

\[ a_s = 0 \]

\[ C_{tem} = 1 \]

\[ C_{dir} = 1 \]

The wind pressure has been represented by a set of storey loads. These forces have been calculated with the Lotus 1-2-3 spreadsheet given in table 1, that performs a simple execution of the above written formulas. These actions were
subsequently applied to each level taking into account that +0.8 corresponds to the upwind face and -0.3 corresponds to the downwind face of the buildings.

**EC3 - 5.2.4.3 F) Frame imperfections loads (FIL)**

*Levels I-II-III-IV-V*

The initial frame imperfections may be represented by a closed system of horizontal forces that are given, for the generic storey (i), by:

\[ F_i = \phi \cdot W_i \]

where:

\[
\phi = \frac{k_c \cdot k_s}{200}
\]

\[
W_i \text{ TOTAL VERTICAL STOREY LOADS}
\]

\[
\phi = \frac{k_c \cdot k_s}{200} \text{ INITIAL SWAY IMPERFECTION}
\]

\[
k_c = \sqrt{0.5 + \frac{1}{n_c}} \quad \text{with } k_c \leq 1
\]

\[
k_s = \sqrt{0.2 + \frac{1}{n_s}} \quad \text{with } k_s \leq 1
\]

in which:

\[ n_c = \text{number of columns} \]

\[ n_s = \text{number of storeys} \]

In our case we have:

\[ n_c = 6 \quad k_c = 0.816 \]

\[ n_s = 5 \quad k_s = 0.632 \]

\[ \phi = \frac{0.816 \cdot 0.632}{200} = 0.00258 \]

In the calculation of the total storey loads, it is necessary to sum to the vertical loads calculated in B) and C) the weight of the frame beams and columns; in our case we have supposed 1.0 KN/m2.

For all the five storeys we obtain:

\[ W = (5+2+1) \cdot 32 \cdot 7.5 = 1920 \text{ KN} \]

The storey equivalent imperfection loads can now be calculated:
These horizontal loads are applied half in one side of the frame, the other half on the other.

III - LOAD CASES

The load combination equations given in the EC1 have been considered:

clause 9.4.5  **A) Ultimate limit states (ULS)**

\[ \sum \gamma_{Gj} \cdot G_{k,i} + 1.35 \cdot \sum Q_{k,i} \]

clause 9.5.5  **B) Serviceability limit states (SLS)**

\[ \sum G_{k,i} + 0.9 \cdot \sum Q_{k,i} \]

where:

- \( G_{k,i} \) = characteristic value of permanent action
- \( Q_{k,i} \) = characteristic value of variable action
- \( \gamma_{Gj} \) = partial factors for permanent actions
The loads considered in this calculation example are:

1) **Permanent actions**

- SWL
- DL
- FIL

2) **Variable actions**

- IL
- SL
- WL

It is important to notice that the equivalent frame imperfections loads have been considered as "permanent" actions since these characterise the geometrical imperfections of the frame elements and thus are a permanent characteristic of the structure.

It is also important to remember that the self weight loads (SWL) have already been increased by 10%.

The **ULS load cases** that have been considered in this calculation example are:

1) \[0.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + 1.35 \cdot FIL\]

2) \[0.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + 1.35 \cdot FIL + 1.35 \cdot WL\]

The **SLS load cases** that have been considered are:

3) \[0.00 \cdot SWL + 1.00 \cdot DL + 0.9 \cdot IL + 1.00 \cdot FIL + 0.9 \cdot WL\]

**IV - PRELIMINARY SIZING OF MEMBERS**
The pre dimensioning of the frame beams and columns has been performed with simple preliminary calculations. The chosen profiles are shown in fig. 3.

EC3 - Chap. 4  **V - SLS CHECKS**

A preliminary first order analysis of the frame has been carried out. In fig. 4 are given the nodal and member numbers. The SLS verifications that have been performed regard horizontal displacement checks of the different storeys of the frame.

According to the EC3, the horizontal deflection limits for multi-storey buildings are:

clause 4.2.2.(4) - *in each storey:*
$$d_i \leq \frac{h_i}{300}$$

- for the structure as a whole:
$$d_{tot} \leq \frac{h}{500}$$

where:
- $d_{tot}$ = total frame horizontal deflection
- $d_i$ = storey (i) relative horizontal deflection
- $h$ = frame total height
- $h_i$ = storey (i) height

The storeys horizontal deflections were obtained calculating the average value between the horizontal displacements of the two end nodes of each storey. It is also necessary to add that the verification of the storey relative horizontal displacement has only been performed in the first storey since it is less stiff than the other levels and thus enables greater displacements.

In our case:

for load combination 3):

- for storey 1:
  $$d_i = 0.639\,\text{cm} \leq \frac{h_i}{300} = \frac{370}{300} = 1.23\,\text{cm}$$

- for the structure as a whole:
  $$d_{tot} = 1.225\,\text{cm} \leq \frac{h}{500} = \frac{1610}{500} = 3.22\,\text{cm}$$
Before the internal forces and moments are calculated and used in member and cross-sectional checks, it is necessary to check if the non linear geometrical effects (second order) have to be taken into account or not.

A frame is classified as "sway" for a given load condition if, for at least one of the storeys, the following condition is satisfied:

\[
\left( \frac{\delta_i}{h_i} \right) \left( \frac{\sum V_i}{\sum H_i} \right) > 0.1
\]

where:
- \( h_i \) = storey\( (i) \) height
- \( \delta_i \) = relative horizontal displacement of storey\( (i) \) obtained from an ordinary first order analysis
- \( \sum V_i \) = total vertical reaction at the bottom of storey\( (i) \) from first order analysis
- \( \sum H_i \) = total horizontal reaction at the bottom of storey\( (i) \) from first order analysis

The values of the "sway-non sway" ratio for the ULS load cases (1 and 2) of this specific example are given in the Lotus 1-2-3- spreadsheets given in tables 2 and 3.

The frame results to be "sway" for both load cases 1 and 2; in these load cases it is necessary to take into account the non linear geometrical effects performing a second order analysis.

section 5.2.4.2(4) **B) CHECK FOR MEMBER IMPERFECTIONS**
According to the EC3, for load cases in which the frame results to be "sway", it is necessary to check whether the effects of the member imperfections can be neglected or not when performing the global analysis of a structure.

For this purpose, if in a compression member with moment resisting connections the given formula is not fulfilled then it is necessary to take into account these imperfections in the global analysis.

\[
\frac{\lambda}{A} \leq 0.5 \cdot \left( \frac{A \cdot f_y}{N_{sd}} \right)
\]

where:

\(\lambda\) = in plane non dimensional slenderness calculated using a buckling length equal to the system length

\(A\) = column cross sectional area
\(N_{sd}\) = design value of the compression force
\(f_y\) = design column yield strength

In the scope of this research project, the above written condition should be checked for all the columns of the frame, taking the internal forces and moments of load cases 1 and 2.

As an example, we can take element 26 (HEA 200) with the internal forces and moments of load combination 1.

In our case:
\(A = 53.8\) cm²
\[ N_{sd} = 589 \text{ KN} \]
\[ f_y = 355 \text{ N/mm}^2 \]
\[ \frac{\lambda}{93.9 \cdot \varepsilon} \cdot \sqrt{\frac{370/8.28}{93.9 \cdot 0.814 \cdot 1}} = 0.585 \]

and thus:
\[ \frac{\lambda}{93.9 \cdot \varepsilon} < 0.5 \cdot \sqrt{\frac{53.8 \cdot 355 \cdot 10^{-1}}{589}} = 0.900 \]

If the above written formula is also fulfilled by the other columns, then is not necessary to take into account the member imperfections in the global analysis of the frame.

Section 5.2.6.2  
**C) SECOND ORDER ANALYSIS**

The EC3 enables to adopt the rigorous second order calculation method together with other indirect second order analysis methods.

5.2.6.2 (2)  
**1) Rigorous second order analysis method**

A rigorous second order analysis has been performed with "PEP micro"; the results (internal forces and moments) obtained with this calculation can be used directly for the local member and cross sectional checks that are performed using the "non sway" buckling lengths.

5.2.6.2 (8)  
**2) Sway mode buckling length method**
This is one of the indirect second order analysis methods indicated by the EC3; for column member and cross-sectional checks we take directly the first order analysis results and adopt "sway mode" buckling lengths; for the beams it is necessary to increase by 20% the "sway moments"; the beams local member and cross-sectional checks are then performed with the increased values of these moments.

5.2.6.2 (3) 3) Amplified sway moments method

This is the other indirect second order calculation method that is applicable only if, for a given load combination, all the storeys of the frame satisfy the following equation:
The "sway moments" that result from the first order analysis have to be increased to take into account the additional effects due to the "sway" behaviour of the frame.

For each different storey, the value of the amplification load factor \( \alpha_i \) is obtained with the formula:

\[
\alpha_i = \frac{1}{1 - \left( \frac{\delta_i}{h_i} \right) \left( \frac{\sum V_i}{\sum H_i} \right)}
\]

In our case:

- for load combination 1)

LEVEL I

\[
\alpha_i = \frac{1}{1 - 0.256} = 1.344
\]

LEVEL II

\[
\alpha_{II} = \frac{1}{1 - 0.114} = 1.129
\]

LEVEL III

\[
\alpha_{III} = \frac{1}{1 - 0.072} = 1.078
\]

LEVEL IV

\[
\alpha_{IV} = \frac{1}{1 - 0.082} = 1.089
\]

LEVEL V

\[
\alpha_{V} = \frac{1}{1 - 0.0414} = 1.043
\]

- for load combination 2)

LEVEL I

\[
\alpha_i = \frac{1}{1 - 0.255} = 1.342
\]
LEVEL II
\[ \alpha_H = \frac{1}{1 - 0.115} = 1.130 \]

LEVEL III
\[ \alpha_{III} = \frac{1}{1 - 0.0735} = 1.079 \]

LEVEL IV
\[ \alpha_{IV} = \frac{1}{1 - 0.0823} = 1.090 \]

LEVEL V
\[ \alpha_V = \frac{1}{1 - 0.0434} = 1.045 \]

It is necessary to observe that for both load cases the value of the "sway-non sway" ratio is slightly greater than 0.25 in the first storey; nevertheless, for the scope of this work, it is interesting to have also the results of a "moment amplification analysis".

In this calculation example the following two methods of applying \( \alpha_i \) have been adopted:

5.2.6.2 (3) a) As it is stated in the EC3, the amplification factors calculated above are used to multiply directly the "sway moments" (that are those moments that result from the horizontal loads and also the vertical loads if either the structure or the loading is asymmetrical). In this way, we obtained incremented values for the bending moments to be used in the local members and cross sectional checks.

This methodology, in the case of symmetrical frames with symmetrical vertical loading, is very simple to adopt (it is in fact sufficient to multiply the part of the moments due to the horizontal loads by the corresponding storey values of \( \alpha_i \)) and gives results that are sufficiently correct.

In fact, the I order moments of storey(i) \( M_{i,i} \) can be expressed in general by:

\[ M_{i,i} = M_i(H) + M_i(V) \]

where:
- \( H = \) horizontal loads
- \( V = \) vertical loads
In the case of symmetrical frames, the second order amplified moments of storey(i) \( M_{ii} \) are calculated by:

\[
M_{ii} = \alpha_i \cdot M_i(H) + M_i(V)
\]

In this calculation example, the horizontal forces are FIL and EL.

b) Some additional calculations have been performed adopting the modified approach described hereafter: instead of multiplying the "sway moments", the factors \( \alpha_i \) were used to multiply the loads that give the horizontal translation of the storeys (i.e. those loads that give the "sway moments").

In our case, according to the definition of "sway moments", it is necessary to multiply only the horizontal loads (i.e. WL and FIL) of each load combination since the frame is symmetric.

As a consequence, for each different storey of the frame we obtain the following load cases with increased partial load factors:

- **load case 1)**

  **LEVEL I**

  \[1.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + (1.344 \cdot 1.35) \cdot FIL\]

  **LEVEL II**

  \[1.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + (1.129 \cdot 1.35) \cdot FIL\]

  **LEVEL III**

  \[1.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + (1.078 \cdot 1.35) \cdot FIL\]

  **LEVEL IV**

  \[1.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + (1.089 \cdot 1.35) \cdot FIL\]

  **LEVEL V**

  \[1.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + (1.043 \cdot 1.35) \cdot FIL\]

- **load case 2)**
LEVEL I
\[ 1.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + (1.342 \cdot 1.35) \cdot FIL + (1.342 \cdot 1.35) \cdot WL \]

LEVEL II
\[ 1.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + (1.130 \cdot 1.35) \cdot FIL + (1.130 \cdot 1.35) \cdot WL \]

LEVEL III
\[ 1.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + (1.079 \cdot 1.35) \cdot FIL + (1.079 \cdot 1.35) \cdot WL \]

LEVEL IV
\[ 1.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + (1.090 \cdot 1.35) \cdot FIL + (1.090 \cdot 1.35) \cdot WL \]

LEVEL V
\[ 1.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + (1.045 \cdot 1.35) \cdot FIL + (1.045 \cdot 1.35) \cdot WL \]

With these new load factors for load cases 1) and 2) a new first order analysis was performed. The internal forces and moments that derive from this new calculation are adopted in the local member and cross-sectional verifications in which we adopt the "non sway mode" buckling lengths.

In tables 4 and 5 it is possible to compare the internal forces of:
- Rigorous second order method;
- Moment amplification method (with \( \alpha_i \) applied to the "sway moments");
- Moment amplification method (with \( \alpha_i \) applied to the loads that give the "sway moments");
- First order method for COLUMNS

- Rigorous second order method;
- Moment amplification method (with \( \alpha_i \) applied to the "sway moments");
- Moment amplification method (with \( \alpha_i \) applied to the loads that give the "sway moments");
- Sway mode buckling lengths method;
- First order method for BEAMS

The comparison between the two indirect second order methods for the columns is only possible comparing the utilisation factors (expressed as a percentage) that are obtained from the bending-axial force buckling member checks.

The following remarks can be made:
columns; if we examine the values of the bending moments, it is visible that those calculated with a rigorous II order analysis are much higher than those of the I order calculation.

In addition, if the two amplification methods are compared with the II order analysis results, it is visible that if we increment directly the "sway moments" (i.e. adopting the method proposed in the EC3) we obtain larger moments; on the other side, if we increment the loads that give the "sway moments" (MAM-F method) we obtain smaller values.

Once the internal forces and moments have been calculated taking into account (when necessary) the second order effects, it is possible to carry out the local member and cross-sectional checks of the elements of the frame.

In this calculation example, only the checks of one column and one beam are illustrated extensively. For the other elements, the computer code developed by A.C.A.I. (Associazione Italiana Costruttori Acciaio) "Eurocode 3", has been used.

The members taken into account, together with their internal forces, are:

- **Column**

  Element 11 (HEA 400), node 14 - Moment amplif. method (MAM-M) - load combination 2

  \[ N = 2980 \text{ kN} \]
  \[ V = 34.3 \text{ kN} \]
  \[ M = 141 \text{ kN*m} \]

- **Beam**

  Element 32 (IPE 500), node 14 - Moment amplif. method - load combination 2

  \[ V = 319 \text{ kN} \]
  \[ M = 510 \text{ kN*m} \]

It is necessary to perform a preliminary operation: the classification of the cross sections of these members. Since an elastic analysis has been performed, the member cross sections may only be considered to be **class 3 or 4** in the local member and cross sectional checks.
A) Column

HEA 400, $f_y = 355$ daN/cm²

1) Web

The formulas for pure compression can be taken into account as a simplification:

- class 1: \[ \frac{d}{t_w} \leq 33 \cdot \varepsilon = 33 \cdot 0.814 = 26.9 \]
- class 2: \[ \frac{d}{t_w} \leq 38 \cdot \varepsilon = 38 \cdot 0.814 = 30.9 \]
- class 3: \[ \frac{d}{t_w} \leq 42 \cdot \varepsilon = 42 \cdot 0.814 = 34.2 \]

In our case:
\[ \frac{d}{t_w} = \frac{298}{11} = 27.1 \Rightarrow \text{class 2} \]

2) Flange

- class 1: \[ \frac{c}{t_f} \leq 10 \cdot \varepsilon = 10 \cdot 0.814 = 8.14 \]
class 2 \[ \frac{c}{t_f} \leq 11 \cdot \varepsilon = 11 \cdot 0.814 = 8.95 \]

class 3 \[ \frac{c}{t_f} \leq 15 \cdot \varepsilon = 15 \cdot 0.814 = 12.2 \]

in our case \[ \frac{c}{t_f} = \frac{150}{19} = 7.89 \Rightarrow \text{class 1} \]

The column can be classified as a class 2 element, but since this is an elastic calculation, the local checks have to be performed according to class 3 rules.

B) Beam

IPE 500, \( f_y = 355 \, \text{daN/cm}^2 \)

1) Web

The formulas for pure bending can be taken into account as a simplification:

class 1 \[ \frac{d}{t_w} \leq 72 \cdot \varepsilon = 72 \cdot 0.814 = 58.6 \]

class 2 \[ \frac{d}{t_w} \leq 83 \cdot \varepsilon = 83 \cdot 0.814 = 67.6 \]

class 3 \[ \frac{d}{t_w} \leq 124 \cdot \varepsilon = 124 \cdot 0.814 = 101 \]

in our case \[ \frac{d}{t_w} = \frac{426}{10.2} = 41.8 \Rightarrow \text{class 1} \]

2) Flange

class 1 \[ \frac{c}{t_f} \leq 10 \cdot \varepsilon = 10 \cdot 0.814 = 8.14 \]

class 2 \[ \frac{c}{t_f} \leq 11 \cdot \varepsilon = 11 \cdot 0.814 = 8.95 \]

class 3 \[ \frac{c}{t_f} \leq 15 \cdot \varepsilon = 15 \cdot 0.814 = 12.2 \]
in our case \[ \frac{c}{t_f} = \frac{100}{16} = 6.25 \Rightarrow \text{class 1} \]

The beam can be classified as a class 1 element, but since this is an elastic calculation, the local checks have to be performed according to \text{class 3} rules.

EC3
sections 5.4 - 5.5

VIII - ULS MEMBER AND CROSS SECTION CHECKS

A) COLUMN

section 5.4.5  
1) Moment cross sectional resistance

For class 3 cross sections:
clause 5.4.5.2
\[ M_{c,Rd} = \frac{W_{el} \cdot f_y}{\gamma_{M0}} \]

In our case:
\[ M_{c,Rd} = \frac{(2311 \cdot 10^{-6}) \cdot (355 \cdot 10^3)}{1.1} = 745.8 \quad > \quad 141 \text{ kN} \cdot \text{m} \]

section 5.4.6  
2) Shear cross sectional resistance

clause 5.4.6(1)
\[ \phi_{l,Rd} = \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} \]
where:

\[ A_v = A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f \]
for I rolled sections
loaded parallel to web

In our case:

\[ V_{pl,Rd} = \frac{[159 - 2 \cdot 30 \cdot 1.9 + (1.1 + 2 \cdot 2.7) \cdot 1.9] \cdot 355 \cdot 10^{-1}}{\sqrt{3} \cdot 1.1} = 1069 > 34.3 \text{ kN} \]

section 5.4.4 3) Compression cross sectional resistance

For class 3 cross sections:

\[ N_{c,Rd} = \frac{A \cdot f_y}{Y_{M0}} \]

In our case:

\[ N_{c,Rd} = \frac{159 \cdot (355 \cdot 10^{-1})}{1.1} = 5131 > 2980 \text{ kN} \]

section 5.4.7 4) Bending and shear cross section resistance

It is not necessary to take into account any type of iteration between M and V since, in our case, we have:

\[ \frac{V_{sd}}{V_{pl,Rd}} = \frac{34.3}{1069} = 0.0321 < 0.5 \]

section 5.4.8 5) Bending and compression cross section resistance

For class 3 sections, the following formula has to be fulfilled:

\[ \frac{N_{sd}}{A \cdot \left( \frac{f_y}{Y_{M0}} \right)} + \frac{M_{sd}}{W_{el} \cdot \left( \frac{f_y}{Y_{M0}} \right)} \leq 1 \]

In our case:

\[ \frac{2980}{159 \cdot \left( \frac{355 \cdot 10^{-1}}{1.1} \right)} + \frac{141}{(2311 \cdot 10^{-6}) \cdot \left( \frac{355 \cdot 10^{3}}{1.1} \right)} = 0.7698 \leq 1 \]
section 5.4.9  

6) Bending, shear and compression cross section resistance

It is not necessary to take into account the iteration between M, V, and N since:

\[ s_d \leq 0.5 \cdot V_{pl,Rd} \]

section 5.6  

7) Shear buckling resistance

This calculation does not have to be performed since:

\[ \frac{298}{11} = 27.1 \leq 0.814 \cdot 69 = 56.2 \]

section 5.5.4  

8) Bending and compression buckling resistance

For members with class 3 cross sections the following formula has to be fulfilled:

\[ \frac{N_{sd}}{\chi \cdot A \cdot \left( \frac{f_y}{\gamma_{M1}} \right) \cdot W_{el} \cdot \left( \frac{f_y}{\gamma_{M1}} \right)} + k \cdot M_{nd} \leq 1 \]

where:

\[ \chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} \quad \text{but} \quad \chi \leq 1 \]

in which:

\[ \phi = 0.5 \left[ 1 + \alpha \cdot (\lambda - 0.2) + \lambda^2 \right] \]

\[ \lambda = \frac{l}{i} = \text{column slenderness} \]

\[ \lambda = 93.9 \cdot \frac{235}{f_y} \]

in which:

\[ \beta_A = 1 \]

\[ \alpha = \text{imperfection factor} \]

\[ k = 1 - \frac{\mu \cdot N_{sd}}{\chi \cdot A \cdot f_y} \quad \text{but} \quad k \leq 1.5 \]

in which:

\[ \mu = \lambda \cdot (2 \cdot \beta_{A} - 4) \quad \text{but} \quad \mu \leq 0.9 \]
where:

$$\beta_M = \text{equivalent uniform moment factor}$$

The buckling lengths "1" have been calculated, both for the "sway" and the "non sway" mode, by the Lotus 1-2-3 spreadsheets given in table 6 and 7. These calculations are a simple application of Annex E of the EC3. In this example, since the internal forces have been calculated using the "indirect second order moment amplification method", the "non sway" buckling lengths have been adopted.

In our case:

$$1 = L \beta$$

$$= 370 \times 0.901 = 333 \text{ cm}$$

$$\lambda = \frac{333}{16.8} = 19.82$$

$$\beta_M = 1.8$$

$$\lambda = \frac{19.82}{\sqrt{93.9 \cdot \frac{235}{355}}} = 0.2594$$

$$\mu = 0.2594 \cdot (2.1.8 - 4) = -0.1038$$

$$\alpha = 0.21 \text{ (buckling curve "a")}$$

$$\phi = 0.5 \left[ 1 + 0.21 \cdot (0.2594 - 0.2) + 0.2594^2 \right] = 0.5399$$

$$\chi = \frac{1}{0.5399 + \sqrt{0.5399^2 - 0.2594^2}} = 0.9868$$

$$k = 1 - \frac{-0.1038 \cdot 2970}{0.9868 \cdot 159 \cdot (355 \cdot 10^{-1})} = 1.055$$

and thus:

$$\frac{2980}{0.9868 \cdot 159 \cdot \left(\frac{355 \cdot 10^{-1}}{L1}\right)} + \frac{1055 \cdot 14100}{2310 \cdot \left(\frac{355 \cdot 10^{-1}}{1.1}\right)} = 0.7880 \leq 1$$

In table 8 it is possible to compare the utilisation ratios, for the M-N buckling check, of some columns calculated with the two indirect second order methods given by the EC3: the "moment amplification method" (with the load amplification factors that multiply directly the "sway moments") and the "sway buckling lengths method". The values of these ratios, for the two methods, are very similar and thus indicate that the degree of precision (for column design) of the two indirect second order methods is approximately the same.
B) BEAM

1) Moment cross sectional resistance

For class 3 cross sections:

\[ M_{c,Rd} = \frac{W_d \cdot f_y}{\gamma_{Mo}} \]

In our case:

\[ M_{c,Rd} = \frac{(1930 \cdot 10^{-6}) \cdot (355 \cdot 10^2)}{1.1} = 622.9 > 510 \text{ kN} \cdot \text{m} \]

2) Shear cross sectional resistance

\[ V_{pl,Rd} = \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{Mo}} \]

where:

\[ A_v = A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f \]

for I rolled sections loaded parallel to web

In our case:

\[ V_{pl,Rd} = \frac{[116 - 2 \cdot 20 \cdot 1.6 + (1.02 + 2 \cdot 2.1) \cdot 1.6] \cdot 355 \cdot 10^{-1}}{\sqrt{3} \cdot 1.1} = 1125 > 319 \text{ kN} \]

3) Bending and shear cross section resistance

It is not necessary to take into account any type of iteration between M and V since, in our case, we have:

\[ \frac{V_{pl,Rd}}{V_{0,Rd}} = \frac{321}{1125} = 0.284 < 0.5 \]

4) Shear buckling resistance

This calculation does not have to be performed since:
\[
\frac{d}{t_w} = \frac{426}{10.2} = 41.8 \leq 0.814 \cdot 69 = 56.2
\]
Multi-storey parking (PAR) analysis
Frame structural model

Note: the dimensions are indicated in millimetres
Multi-storey parking (PAR) analysis
Parking areas and circulation ways layout

Note: the dimensions are indicated in metres

ECSC Project 7210-SA/419
Practical application of EUROCODE 3
to multi-storey buildings with steel
"Sway Frame" structures
Multi-storey parking (PAR) non seismic analysis
Member profiles

ECSC Project 7210-SA/419
Practical application of EUROCODE 3
to multi-storey buildings with steel "Sway Frame" structures
### Node numbers

<p>| | | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>12</td>
<td>18</td>
<td>24</td>
<td>30</td>
<td>36</td>
</tr>
<tr>
<td>5</td>
<td>11</td>
<td>17</td>
<td>23</td>
<td>29</td>
<td>35</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
<td>16</td>
<td>22</td>
<td>28</td>
<td>34</td>
</tr>
<tr>
<td>3</td>
<td>8</td>
<td>14</td>
<td>20</td>
<td>26</td>
<td>32</td>
</tr>
</tbody>
</table>

### Element numbers

<p>| | | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>10</td>
<td>15</td>
<td>20</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>4</td>
<td>8</td>
<td>14</td>
<td>19</td>
<td>24</td>
<td>29</td>
</tr>
<tr>
<td>3</td>
<td>9</td>
<td>13</td>
<td>18</td>
<td>23</td>
<td>28</td>
</tr>
<tr>
<td>2</td>
<td>11</td>
<td>12</td>
<td>17</td>
<td>22</td>
<td>27</td>
</tr>
<tr>
<td>1</td>
<td>5</td>
<td>16</td>
<td>21</td>
<td>26</td>
<td>31</td>
</tr>
</tbody>
</table>

**FIGURE 4**
### EUROCODE 1: WIND ACTIONS

**Title:** Multi-storey parking building (PAR) calculation example

| C_t | building diagonal length | 62.1 (m) |
| C_DIR | direction factor | 1 |
| C_TEM | seasonal factor | 1 |
| a_s | height above sea level | 0 (Km) |
| K_a | a_s coefficient (see 2.7.A.9.11) | 0.55 (1/Km) |
| Vref,0 | reference wind speed for a_s=0 | 28 (m/s) |
| B | distance between transversal frames | 7.5 (m) |
| K_r | roughness factor | 0.24 |
| z_0 | roughness length | 1 (m) |
| z_min | minimum height | 16 (m) |

<table>
<thead>
<tr>
<th>Level</th>
<th>height</th>
<th>reference elevation</th>
<th>positive pressure coefficient</th>
<th>negative pressure coefficient</th>
<th>positive storey load</th>
<th>negative storey load</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>3.4</td>
<td>3.7</td>
<td>0.8</td>
<td>0.3</td>
<td>10.52</td>
<td>3.95</td>
</tr>
<tr>
<td>II</td>
<td>3.1</td>
<td>6.8</td>
<td>0.8</td>
<td>0.3</td>
<td>9.59</td>
<td>3.60</td>
</tr>
<tr>
<td>III</td>
<td>3.1</td>
<td>9.9</td>
<td>0.8</td>
<td>0.3</td>
<td>9.59</td>
<td>3.60</td>
</tr>
<tr>
<td>IV</td>
<td>3.1</td>
<td>13</td>
<td>0.8</td>
<td>0.3</td>
<td>9.59</td>
<td>3.60</td>
</tr>
<tr>
<td>V</td>
<td>1.55</td>
<td>16.1</td>
<td>0.8</td>
<td>0.3</td>
<td>4.81</td>
<td>1.80</td>
</tr>
<tr>
<td>VI</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>VII</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>VIII</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>VIII</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>IX</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**TABLE 1**
<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (cm)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
<td>579</td>
<td>3.630</td>
<td>0.000</td>
<td>2</td>
<td>0.237</td>
<td>0.237</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>2250</td>
<td>16.500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>13</td>
<td>2980</td>
<td>-1.500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>19</td>
<td>2970</td>
<td>-6.690</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>25</td>
<td>2260</td>
<td>29.100</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>31</td>
<td>592</td>
<td>5.530</td>
<td>0.000</td>
<td>32</td>
<td>0.295</td>
<td>0.295</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>11631</td>
<td>32.690</td>
<td>0.000</td>
<td>370</td>
<td>mean rel.</td>
<td>0.266</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sway criterion:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storey</td>
<td>Load Comb.</td>
<td>Lower nodes</td>
<td>N (KN)</td>
<td>V (KN)</td>
<td>total disp. (cm)</td>
<td>Upper nodes</td>
<td>total disp. (cm)</td>
<td>relative disp. (cm)</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
<td>-------------</td>
<td>--------</td>
<td>--------</td>
<td>------------------</td>
<td>-------------</td>
<td>------------------</td>
<td>-------------------</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>2</td>
<td>460</td>
<td>15.000</td>
<td>0.237</td>
<td>3</td>
<td>0.361</td>
<td>0.124</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>1800</td>
<td>58.100</td>
<td></td>
<td>9</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>14</td>
<td>2380</td>
<td>-14.100</td>
<td></td>
<td>13</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>20</td>
<td>2380</td>
<td>-0.125</td>
<td></td>
<td>21</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>26</td>
<td>1800</td>
<td>-9.700</td>
<td></td>
<td>27</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>32</td>
<td>467</td>
<td>-17.500</td>
<td>0.295</td>
<td>33</td>
<td>0.371</td>
<td>0.076</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>9287</td>
<td>-26.225</td>
<td>0.000</td>
<td>310</td>
<td>mean rel.</td>
<td>0.100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sway criterion:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storey</td>
<td>Load Comb.</td>
<td>Lower nodes</td>
<td>N (KN)</td>
<td>V (KN)</td>
<td>total disp. (cm)</td>
<td>Upper nodes</td>
<td>total disp. (cm)</td>
<td>relative disp. (cm)</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
<td>-------------</td>
<td>--------</td>
<td>--------</td>
<td>------------------</td>
<td>-------------</td>
<td>------------------</td>
<td>-------------------</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>3</td>
<td>337</td>
<td>16.900</td>
<td>0.361</td>
<td>4</td>
<td>0.449</td>
<td>0.088</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>1350</td>
<td>66.600</td>
<td></td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>15</td>
<td>1790</td>
<td>-13.200</td>
<td></td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>21</td>
<td>1790</td>
<td>2.220</td>
<td></td>
<td>22</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>27</td>
<td>1360</td>
<td>-7.800</td>
<td></td>
<td>28</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>33</td>
<td>361</td>
<td>-18.400</td>
<td>0.371</td>
<td>34</td>
<td>0.409</td>
<td>0.038</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>6968</td>
<td>-19.680</td>
<td>0.000</td>
<td>310</td>
<td>mean rel.</td>
<td>0.063</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sway criterion:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storey</td>
<td>Load Comb.</td>
<td>Lower nodes</td>
<td>N (KN)</td>
<td>V (KN)</td>
<td>total disp. (cm)</td>
<td>Upper nodes</td>
<td>total disp. (cm)</td>
<td>relative disp. (cm)</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
<td>-------------</td>
<td>--------</td>
<td>--------</td>
<td>------------------</td>
<td>-------------</td>
<td>------------------</td>
<td>-------------------</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>4</td>
<td>217</td>
<td>8.940</td>
<td>0.449</td>
<td>5</td>
<td>0.497</td>
<td>0.048</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10</td>
<td>906</td>
<td>39.700</td>
<td></td>
<td>11</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>1190</td>
<td>7.250</td>
<td></td>
<td>17</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>22</td>
<td>1190</td>
<td>0.667</td>
<td></td>
<td>23</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>28</td>
<td>907</td>
<td>45.200</td>
<td></td>
<td>29</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>34</td>
<td>219</td>
<td>-9.890</td>
<td>0.409</td>
<td>35</td>
<td>0.504</td>
<td>0.095</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>4629</td>
<td>-13.013</td>
<td>0.000</td>
<td>310</td>
<td>mean rel.</td>
<td>0.072</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sway criterion:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storey</td>
<td>Load Comb.</td>
<td>Lower nodes</td>
<td>N (KN)</td>
<td>V (KN)</td>
<td>total disp. (cm)</td>
<td>Upper nodes</td>
<td>total disp. (cm)</td>
<td>relative disp. (cm)</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
<td>-------------</td>
<td>--------</td>
<td>--------</td>
<td>------------------</td>
<td>-------------</td>
<td>------------------</td>
<td>-------------------</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>5</td>
<td>103</td>
<td>8.710</td>
<td>0.497</td>
<td>6</td>
<td>0.568</td>
<td>0.071</td>
</tr>
<tr>
<td></td>
<td></td>
<td>11</td>
<td>457</td>
<td>51.300</td>
<td></td>
<td>12</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>17</td>
<td>599</td>
<td>10.700</td>
<td></td>
<td>18</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>23</td>
<td>599</td>
<td>7.290</td>
<td></td>
<td>24</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>29</td>
<td>457</td>
<td>53.900</td>
<td></td>
<td>30</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>35</td>
<td>103</td>
<td>-9.200</td>
<td>0.504</td>
<td>36</td>
<td>0.555</td>
<td>0.001</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>2318</td>
<td>-6.500</td>
<td>0.000</td>
<td>310</td>
<td>mean rel.</td>
<td>0.036</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sway criterion:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 2**
### TABLE 3

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (cm)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>1</td>
<td>563</td>
<td>12.240</td>
<td>0.000</td>
<td>2</td>
<td>0.904</td>
<td>0.904</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>2230</td>
<td>0.655</td>
<td></td>
<td>6</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>13</td>
<td>2980</td>
<td>-3.200</td>
<td></td>
<td>14</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>19</td>
<td>2970</td>
<td>-2.900</td>
<td></td>
<td>20</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>25</td>
<td>2270</td>
<td>0.000</td>
<td></td>
<td>26</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>31</td>
<td>607</td>
<td>-7.900</td>
<td>0.000</td>
<td>32</td>
<td>0.954</td>
<td>0.954</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>11620</td>
<td>-114.505</td>
<td>Storey height</td>
<td>370</td>
<td>mean rel. disp. (cm)</td>
<td>0.929</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sway criterion:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(N tot x st.disp.) / (h x V lot) =</td>
<td>0.000</td>
<td>0.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (cm)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>2</td>
<td>2</td>
<td>452</td>
<td>12.100</td>
<td>0.904</td>
<td>3</td>
<td>1.270</td>
<td>0.366</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>1790</td>
<td>46.900</td>
<td></td>
<td>9</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>14</td>
<td>2380</td>
<td>-31.000</td>
<td></td>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>20</td>
<td>2380</td>
<td>-17.100</td>
<td></td>
<td>21</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>26</td>
<td>1810</td>
<td>-78.900</td>
<td></td>
<td>27</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>32</td>
<td>475</td>
<td>-20.500</td>
<td>0.954</td>
<td>33</td>
<td>1.270</td>
<td>0.316</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>9287</td>
<td>-88.500</td>
<td>Storey height</td>
<td>310</td>
<td>mean rel. disp. (cm)</td>
<td>0.341</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sway criterion:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(N tot x st.disp.) / (h x V lot) =</td>
<td>0.000</td>
<td>0.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (cm)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>2</td>
<td>3</td>
<td>233</td>
<td>15.100</td>
<td>1.270</td>
<td>4</td>
<td>1.500</td>
<td>0.220</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>1350</td>
<td>58.500</td>
<td></td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>15</td>
<td>1790</td>
<td>-25.600</td>
<td></td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>21</td>
<td>1790</td>
<td>-10.200</td>
<td></td>
<td>22</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>27</td>
<td>1360</td>
<td>-81.900</td>
<td></td>
<td>28</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>33</td>
<td>344</td>
<td>-20.100</td>
<td>1.270</td>
<td>34</td>
<td>1.460</td>
<td>0.190</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>6967</td>
<td>-64.200</td>
<td>Storey height</td>
<td>310</td>
<td>mean rel. disp. (cm)</td>
<td>0.210</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sway criterion:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(N tot x st.disp.) / (h x V lot) =</td>
<td>0.000</td>
<td>0.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (cm)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>2</td>
<td>4</td>
<td>215</td>
<td>7.960</td>
<td>1.500</td>
<td>5</td>
<td>1.700</td>
<td>0.200</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10</td>
<td>905</td>
<td>34.000</td>
<td></td>
<td>11</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>1190</td>
<td>-14.000</td>
<td></td>
<td>17</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>22</td>
<td>1190</td>
<td>-6.010</td>
<td></td>
<td>23</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>28</td>
<td>908</td>
<td>-50.900</td>
<td></td>
<td>29</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>34</td>
<td>220</td>
<td>-10.900</td>
<td>1.460</td>
<td>35</td>
<td>1.700</td>
<td>0.240</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>4628</td>
<td>-39.850</td>
<td>Storey height</td>
<td>310</td>
<td>mean rel. disp. (cm)</td>
<td>0.220</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sway criterion:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(N tot x st.disp.) / (h x V lot) =</td>
<td>0.000</td>
<td>0.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (cm)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>2</td>
<td>5</td>
<td>103</td>
<td>8.400</td>
<td>1.700</td>
<td>6</td>
<td>1.820</td>
<td>0.120</td>
</tr>
<tr>
<td></td>
<td></td>
<td>11</td>
<td>456</td>
<td>49.600</td>
<td></td>
<td>12</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>17</td>
<td>599</td>
<td>-13.100</td>
<td></td>
<td>18</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>23</td>
<td>599</td>
<td>4.850</td>
<td></td>
<td>24</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>29</td>
<td>457</td>
<td>-55.700</td>
<td></td>
<td>30</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>35</td>
<td>104</td>
<td>3.540</td>
<td>1.700</td>
<td>36</td>
<td>1.760</td>
<td>0.060</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>2318</td>
<td>-15.490</td>
<td>Storey height</td>
<td>310</td>
<td>mean rel. disp. (cm)</td>
<td>0.090</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sway criterion:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(N tot x st.disp.) / (h x V lot) =</td>
<td>0.000</td>
<td>0.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Type of Analysis

<table>
<thead>
<tr>
<th>Order</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>First order analysis</td>
</tr>
<tr>
<td>2 - MAM-M</td>
<td>Indirect second order analysis - Moment amplification method; amplification factors applied to the loads that give the &quot;sway moments&quot;</td>
</tr>
<tr>
<td>2 - MAM-F</td>
<td>Indirect second order analysis - Moment amplification method; amplification factors applied to &quot;sway moments&quot;</td>
</tr>
<tr>
<td>2 - SMBL</td>
<td>Indirect second order analysis - Sway mode buckling lengths method</td>
</tr>
<tr>
<td>2</td>
<td>Rigorous second order analysis</td>
</tr>
</tbody>
</table>

### INTERNAL FORCES AND MOMENTS

<table>
<thead>
<tr>
<th>Elem.</th>
<th>Node</th>
<th>Comb.</th>
<th>I order</th>
<th>II order - MAM-F</th>
<th>II order - MAM-M</th>
<th>II order - SMBL</th>
<th>II order</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Num.</td>
<td>Num.</td>
<td>Num.</td>
<td>N (kN)</td>
<td>V (kN)</td>
<td>M (kN·m)</td>
<td>N (kN)</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>2</td>
<td></td>
<td>561</td>
<td>-1.24</td>
<td>4.59</td>
<td>559</td>
</tr>
<tr>
<td>6</td>
<td>8</td>
<td>2</td>
<td></td>
<td>2230</td>
<td>-0.81</td>
<td>3.00</td>
<td>2230</td>
</tr>
<tr>
<td>11</td>
<td>14</td>
<td>2</td>
<td></td>
<td>2980</td>
<td>34.3</td>
<td>-127</td>
<td>2980</td>
</tr>
<tr>
<td>16</td>
<td>20</td>
<td>2</td>
<td></td>
<td>2960</td>
<td>29.5</td>
<td>-109</td>
<td>2960</td>
</tr>
<tr>
<td>21</td>
<td>26</td>
<td>2</td>
<td></td>
<td>2260</td>
<td>-4.8</td>
<td>-166</td>
<td>2260</td>
</tr>
<tr>
<td>26</td>
<td>32</td>
<td>2</td>
<td></td>
<td>605</td>
<td>7.90</td>
<td>-0.29</td>
<td>608</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>2</td>
<td></td>
<td>451</td>
<td>-12.0</td>
<td>18.1</td>
<td>450</td>
</tr>
<tr>
<td>17</td>
<td>21</td>
<td>2</td>
<td></td>
<td>2370</td>
<td>15.8</td>
<td>-31.6</td>
<td>2370</td>
</tr>
<tr>
<td>27</td>
<td>33</td>
<td>2</td>
<td></td>
<td>474</td>
<td>20.6</td>
<td>-31.9</td>
<td>475</td>
</tr>
</tbody>
</table>
### EC3: "SWAY FRAMES" ANALYSIS RESULTS

Title: Multi storey parking (PARK) - Non seismic design - BEAMS

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>31</td>
<td>2</td>
<td>2</td>
<td>109</td>
<td>-23.9</td>
<td>107</td>
<td>109</td>
<td>106</td>
</tr>
<tr>
<td>31</td>
<td>8</td>
<td>2</td>
<td>-177</td>
<td>-161</td>
<td>-179</td>
<td>-177</td>
<td>-180</td>
</tr>
<tr>
<td>32</td>
<td>8</td>
<td>2</td>
<td>259</td>
<td>-244</td>
<td>256</td>
<td>259</td>
<td>254</td>
</tr>
<tr>
<td>32</td>
<td>14</td>
<td>2</td>
<td>-319</td>
<td>-484</td>
<td>-321</td>
<td>-319</td>
<td>-324</td>
</tr>
<tr>
<td>33</td>
<td>14</td>
<td>2</td>
<td>271</td>
<td>-316</td>
<td>268</td>
<td>271</td>
<td>266</td>
</tr>
<tr>
<td>33</td>
<td>20</td>
<td>2</td>
<td>-307</td>
<td>-461</td>
<td>-309</td>
<td>-307</td>
<td>-312</td>
</tr>
<tr>
<td>34</td>
<td>20</td>
<td>2</td>
<td>281</td>
<td>-335</td>
<td>278</td>
<td>281</td>
<td>276</td>
</tr>
<tr>
<td>34</td>
<td>26</td>
<td>2</td>
<td>-297</td>
<td>-398</td>
<td>-299</td>
<td>-297</td>
<td>-301</td>
</tr>
<tr>
<td>35</td>
<td>26</td>
<td>2</td>
<td>157</td>
<td>-115</td>
<td>155</td>
<td>157</td>
<td>154</td>
</tr>
<tr>
<td>35</td>
<td>32</td>
<td>2</td>
<td>-129</td>
<td>-60.9</td>
<td>-131</td>
<td>-129</td>
<td>-132</td>
</tr>
<tr>
<td>39</td>
<td>21</td>
<td>2</td>
<td>290</td>
<td>-371</td>
<td>289</td>
<td>290</td>
<td>289</td>
</tr>
<tr>
<td>39</td>
<td>27</td>
<td>2</td>
<td>-288</td>
<td>-365</td>
<td>-289</td>
<td>-288</td>
<td>-289</td>
</tr>
<tr>
<td>40</td>
<td>27</td>
<td>2</td>
<td>157</td>
<td>-118</td>
<td>156</td>
<td>157</td>
<td>156</td>
</tr>
<tr>
<td>40</td>
<td>33</td>
<td>2</td>
<td>-129</td>
<td>-61.8</td>
<td>-130</td>
<td>-129</td>
<td>-130</td>
</tr>
</tbody>
</table>

### INTERNAL FORCES AND MOMENTS

<table>
<thead>
<tr>
<th>N (kN)</th>
<th>V (kN)</th>
<th>M (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>109</td>
<td>-23.9</td>
<td>107</td>
</tr>
<tr>
<td>107</td>
<td>-21.2</td>
<td>109</td>
</tr>
<tr>
<td>-177</td>
<td>-161</td>
<td>-179</td>
</tr>
<tr>
<td>-179</td>
<td>-164</td>
<td>-177</td>
</tr>
<tr>
<td>259</td>
<td>-244</td>
<td>256</td>
</tr>
<tr>
<td>256</td>
<td>-233</td>
<td>259</td>
</tr>
<tr>
<td>-319</td>
<td>-484</td>
<td>-321</td>
</tr>
<tr>
<td>-321</td>
<td>-494</td>
<td>-319</td>
</tr>
<tr>
<td>271</td>
<td>-316</td>
<td>268</td>
</tr>
<tr>
<td>268</td>
<td>-306</td>
<td>271</td>
</tr>
<tr>
<td>-307</td>
<td>-461</td>
<td>-309</td>
</tr>
<tr>
<td>-309</td>
<td>-471</td>
<td>-307</td>
</tr>
<tr>
<td>281</td>
<td>-335</td>
<td>278</td>
</tr>
<tr>
<td>278</td>
<td>-325</td>
<td>281</td>
</tr>
<tr>
<td>-297</td>
<td>-398</td>
<td>-299</td>
</tr>
<tr>
<td>-299</td>
<td>-409</td>
<td>-297</td>
</tr>
<tr>
<td>157</td>
<td>-115</td>
<td>155</td>
</tr>
<tr>
<td>155</td>
<td>-112</td>
<td>157</td>
</tr>
<tr>
<td>-129</td>
<td>-60.9</td>
<td>-131</td>
</tr>
<tr>
<td>-131</td>
<td>-63.6</td>
<td>-129</td>
</tr>
<tr>
<td>290</td>
<td>-371</td>
<td>289</td>
</tr>
<tr>
<td>289</td>
<td>-361</td>
<td>290</td>
</tr>
<tr>
<td>-288</td>
<td>-365</td>
<td>-289</td>
</tr>
<tr>
<td>-289</td>
<td>-372</td>
<td>-288</td>
</tr>
<tr>
<td>157</td>
<td>-118</td>
<td>156</td>
</tr>
<tr>
<td>156</td>
<td>-116</td>
<td>157</td>
</tr>
<tr>
<td>-129</td>
<td>-61.8</td>
<td>-130</td>
</tr>
<tr>
<td>-130</td>
<td>-63.2</td>
<td>-129</td>
</tr>
</tbody>
</table>

### TYPE OF ANALYSIS

- **I order**: First order analysis
- **II order - MAM-F**: Indirect second order analysis - Moment amplification method; amplification factors applied to the loads that give the "sway moments"
- **II order - MAM-M**: Indirect second order analysis - Moment amplification method; amplification factors applied to "sway moments"
- **II order - SMBL**: Indirect second order analysis - Sway mode buckling lengths method
- **II order**: Rigorous second order analysis

**TABLE 5**
Title: Multi storey parking building (PAR) non seismic calculation example

### TABLE 6

<table>
<thead>
<tr>
<th>COLUMN NUMBER</th>
<th>DEFL. MODE</th>
<th>COLUMN N</th>
<th>COLUMNS</th>
<th>BEAMS</th>
<th>BETA (l/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>KC K1 K2</td>
<td>gA K A KA</td>
<td>gB K B KB</td>
<td>gC K C KC</td>
</tr>
<tr>
<td>1</td>
<td>N</td>
<td>9.98 8.1</td>
<td>0 0 0</td>
<td>0.5 20.9 10.4</td>
<td>0 0 0</td>
</tr>
<tr>
<td>2</td>
<td>N</td>
<td>8.1 8.1 9.98</td>
<td>0 0 0</td>
<td>0.5 20.9 10.4</td>
<td>0 0 0</td>
</tr>
<tr>
<td>3</td>
<td>N</td>
<td>8.1 3.33 8.1</td>
<td>0 0 0</td>
<td>0.5 20.9 10.4</td>
<td>0 0 0</td>
</tr>
<tr>
<td>4</td>
<td>N</td>
<td>3.33 3.33 8.1</td>
<td>0 0 0</td>
<td>0.5 20.9 10.4</td>
<td>0 0 0</td>
</tr>
<tr>
<td>5</td>
<td>N</td>
<td>3.33 0 3.33</td>
<td>0 0 0</td>
<td>0.5 20.9 10.4</td>
<td>0 0 0</td>
</tr>
<tr>
<td>11</td>
<td>N</td>
<td>122 74 0</td>
<td>0.5 60.2 30.1</td>
<td>0.5 60.2 30.1</td>
<td>0 0 0</td>
</tr>
<tr>
<td>12</td>
<td>N</td>
<td>74 74 122</td>
<td>0.5 60.2 30.1</td>
<td>0.5 60.2 30.1</td>
<td>0.5 60.2 30.1</td>
</tr>
<tr>
<td>13</td>
<td>N</td>
<td>74 25 74</td>
<td>0.5 60.2 30.1</td>
<td>0.5 60.2 30.1</td>
<td>0.5 60.2 30.1</td>
</tr>
<tr>
<td>14</td>
<td>N</td>
<td>25 25 74</td>
<td>0.5 60.2 30.1</td>
<td>0.5 60.2 30.1</td>
<td>0.5 60.2 30.1</td>
</tr>
<tr>
<td>15</td>
<td>N</td>
<td>25 0 25</td>
<td>0.5 60.2 30.1</td>
<td>0.5 60.2 30.1</td>
<td>0.5 60.2 30.1</td>
</tr>
</tbody>
</table>

Values of "g" factors (for beams):
- fixed at far end: 1
- pinned at far end: 0.75
- single curvature: 0.5
- double curvature: 1.5

Values of "n" for each column end:
- fixed end: 0
- pinned end: 1
<table>
<thead>
<tr>
<th>COLUMN NUMBER</th>
<th>DEFL. MODE</th>
<th>COLUMNS</th>
<th>BEAMS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>KC</td>
<td>K1</td>
<td>K2</td>
</tr>
<tr>
<td>1</td>
<td>S</td>
<td>9.98</td>
<td>8.1</td>
</tr>
<tr>
<td>2</td>
<td>S</td>
<td>8.1</td>
<td>8.1</td>
</tr>
<tr>
<td>3</td>
<td>S</td>
<td>8.1</td>
<td>3.33</td>
</tr>
<tr>
<td>4</td>
<td>S</td>
<td>3.33</td>
<td>3.33</td>
</tr>
<tr>
<td>5</td>
<td>S</td>
<td>3.33</td>
<td>0</td>
</tr>
<tr>
<td>11</td>
<td>S</td>
<td>122</td>
<td>74</td>
</tr>
<tr>
<td>12</td>
<td>S</td>
<td>74</td>
<td>74</td>
</tr>
<tr>
<td>13</td>
<td>S</td>
<td>74</td>
<td>25</td>
</tr>
<tr>
<td>14</td>
<td>S</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>15</td>
<td>S</td>
<td>25</td>
<td>0</td>
</tr>
</tbody>
</table>

Values of "g" factors (for beams):
* fixed at far end: 1
* pinned at far end: 0.75
* single curvature: 0.5
* double curvature: 1.5

Values of "n" for each column end:
* fixed end: 0
* pinned end: 1

TABLE 7
<table>
<thead>
<tr>
<th>COLUMN NUMBER</th>
<th>LOAD COMB. RATIO</th>
<th>BENDING-COMPRESSION BUCKLING CHECK RATIO</th>
<th>AMPLIFIED SWAY MOMENTS METHOD</th>
<th>SWAY MODE BUCKLING LENGTHS METHOD</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>2</td>
<td>0.8919</td>
<td></td>
<td>0.8901</td>
</tr>
<tr>
<td>12</td>
<td>2</td>
<td>0.8542</td>
<td></td>
<td>0.8496</td>
</tr>
<tr>
<td>13</td>
<td>2</td>
<td>0.6447</td>
<td></td>
<td>0.6427</td>
</tr>
<tr>
<td>14</td>
<td>2</td>
<td>0.7659</td>
<td></td>
<td>0.7654</td>
</tr>
<tr>
<td>15</td>
<td>2</td>
<td>0.4318</td>
<td></td>
<td>0.4329</td>
</tr>
</tbody>
</table>

**TABLE 8**
MULTI-STOREY PARKING BUILDING (PAR)

SEISMIC CALCULATION
Type of building : Multi-storey parking (PAR)
Type of design : Seismic

BUILDING AND STRUCTURAL DATA

- **Dimensions**
  - Length (m) : 60
  - Width (m) : 32
  - Height (m) : 16.1

- **Storeys**
  - Number : 5

- **Structural system**
  - Transversal direction: the bracing system for the horizontal loads is given by nine beam-column rigid jointed frames whose distance is 7.5 m. The columns of these structures are supposed to be hinged at the bases.
  - Longitudinal direction: the bracing system is obtained with a number of cross bracing structures that are added to the frame that is supposed to have beam-column hinged joints.

MATERIAL

- Steel Grade (for profiles, plates and welds) : Fe 510
- Bolts grade : 8.8

SEISMIC DESIGN

- Ground acceleration : \( \frac{a}{g} = 0.15 \)
- Behaviour factor : 6
I - GENERAL INFORMATION

The structural model of the frame is given in fig.1. The geometry of the structure and the position of the columns was decided taking into account the layout of the parking areas and the circulation ways that is shown, as a typical solution for all the storeys, in fig.2.

It is important to observe that, for "q = 6", all the frame members must have class 1 cross sections. In this calculation example, all the adopted profiles fulfil this indirect ductility requirement.

The described calculations refer to one of the nine transversal frames of the building; in this way we refer to a simplified two dimensional model of the structure. The joints between the beams and the columns were supposed to be full strength connections and the loads that have been considered refer to a portion of building whose width is equal to the distance between the frames (thus, in this case, we take into account a portion of 7.5 m).

To facilitate the comprehension of the calculations, at the beginning of each section is reproduced a copy of the general flow chart "FC1" in which is evidenced the design phase that is currently being carried out.

II - LOADS

The following loads have been taken into account:

A) Self weight loads (SWL)
These include the weight of the steel frame members only and are calculated automatically by the codes used in this calculation example ("Stranger" and "PEP micro").

To take into account the weight of the joint elements (i.e. end plates, stiffeners, welds, bolts) these elementary self weight loads have been increased by 10% in the analysis and thus a $\lambda = 1.1$ load factor is always adopted.

**B) Dead loads (DL)**

**Levels I-II-III-IV-V**

- r.c. slab
- secondary steel members self weight
- finishing coat

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>r.c. slab</td>
<td>4</td>
</tr>
<tr>
<td>secondary steel members self weight</td>
<td>0.5</td>
</tr>
<tr>
<td>finishing coat</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Total uniform loads

Total uniform load for 7.5 m wide building portion

\[ 7.5 \times 5 = 37.5 \text{ KN/m} \]

**C) Imposed loads (IL)**

**Levels I-II-III-IV (and also V if these are greater than the snow loads)**

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>category F</td>
<td>2</td>
</tr>
</tbody>
</table>

Total uniform load for 7.5 m building portion

\[ 2 \times 7.5 = 15 \text{ KN/m} \]

**D) Snow loads (SL)**

**Level V**

The snow loads are given by:

\[ S = \mu_i \cdot C_e \cdot C_i \cdot S_k \]

In our case:

- $\mu_i = 0.8$ (monopitch roof with $\alpha = 0$)
- $C_e = 1$
- $C_i = 1$
- $S_k = 90 \text{ KN/m}^2$ (Genova, Italy, $A = 0$)
\[ S = 0.8 \cdot 1 \cdot 0.9 = 0.72 \text{ KN/m}^2 \]

It is important to add that in this case the imposed loads have been considered for the V storey, since these have a higher value than the snow loads.

ECI part 2.1  
Chapter 6

E) Wind loads (WL)

The pressure acting on the external surfaces is:

\[ w_e(z) = c_p \cdot c_{size} \cdot c_e(z) \cdot q_{ref} \]

in which:

\[ c_{size} = 0.49 \left[ 1 + 0.063 \cdot \ln \left( \frac{3300}{l} \right) \right]^2 \quad \text{SIZE COEFFICIENT} \]

\[ c_e(z) = c_e(z) \cdot c_i(z)^2 \cdot \left[ 1 + \frac{1.4}{c_e(z) \cdot c_i(z)} \right] \quad \text{EXPOSURE COEFFICIENT} \]

\[ c_r(z) = c_r(z_{\text{min}}) \quad \text{for } 0 < z < z_{\text{min}} \]

\[ c_r(z) = K_r \cdot \ln \left( \frac{z}{z_0} \right) \quad \text{for } z > z_{\text{min}} \quad \text{ROUGHNESS COEFFICIENT} \]

\[ C_t = 1 \quad \text{for flat terrains} \quad \text{TOPOGRAPHY COEFFICIENT} \]

\[ q_{ref} = \frac{\rho}{2} \cdot v_{ref}^2 \quad \text{WIND PRESSURE} \]

in which:

\[ v_{ref} = v_{ref,0} \cdot (1 + K_a \cdot c_s) \cdot c_{top} \cdot c_{dir} \]

\[ c_p \quad \text{PRESSURE COEFFICIENT} \]

In our case, the input parameters are:

annex B - 1.2  

\[ c_p = +0.8 \quad \text{Upwind face} \]
\[ = -0.3 \quad \text{Downwind face} \]

\[ l = \text{diagonal size of loaded area} \]
\[ = \sqrt{16.1^2 + 60^2} = 62 \quad \text{m} \]

annex A - 4  

terrain roughness: type IV
\[ K_r = 0.24 \]
\[ z_0 = 1 \quad \text{m} \]
Zmin = 16 m

\[ \rho = 1.25 \text{ Kg/m}^3 \]

\[ \text{Vref}_0 = 28 \text{ m/s} \quad \text{(for Genova, Italy, cat.3)} \]

\[ K_a = 0.55 \]

\[ a_s = 0 \]

\[ C_{tem} = 1 \]

\[ C_{dir} = 1 \]

The wind pressure has been represented by a set of storey loads. These forces have been calculated with the Lotus 1-2-3 spreadsheet given in table 1, that performs a simple execution of the above written formulas. These actions were subsequently applied to each level taking into account that +0.8 corresponds to the upwind face and -0.3 corresponds to the downwind face of the buildings.

**EC3 - 5.2.4.3**

**F) Frame imperfections loads (FIL)**

**Levels I-II-III-IV-V**

The initial frame imperfections may be represented by a closed system of horizontal forces that are given, for the generic storey \( i \), by:

\[ F_i = \phi \cdot W_i \]

where:

\[ \phi = \frac{k_s \cdot k_c}{200} \]

\[ W_i \quad \text{TOTAL VERTICAL STOREY LOADS} \]

\[ \phi \quad \text{INITIAL SWAY IMPERFECTION} \]

\[ k_c = \sqrt{0.5 + \frac{1}{n_c}} \quad \text{with } k_c \leq 1 \]

\[ k_s = \sqrt{0.2 + \frac{1}{n_s}} \quad \text{with } k_s \leq 1 \]

in which:

\( n_c = \text{number of columns} \)

\( n_s = \text{number of storeys} \)

In our case we have:

\( n_c = 6 \quad k_c = 0.816 \)

\( n_s = 5 \quad k_s = 0.632 \)
In the calculation of the total storey loads, it is necessary to sum to the vertical loads calculated in B) and C) the weight of the frame beams and columns; in our case we have supposed 1.0 KN/m².

For all the five storeys we obtain:

\[ W = (5+2+1) \times 32 \times 7.5 = 1920 \, KN \]

The storey equivalent imperfection loads can now be calculated:

\[ F = 0.00258 \times 1920 = 4.95 \, KN \]

These horizontal loads are applied half in one side of the frame, the other half on the other.

EC8

\( G \) Earthquake loads (EL)

The following input data is considered:

- Frame natural period \( T_0 = 1.2 \, s \)
- Structural viscous damping ratio \( \zeta = 5\% \)
- Behaviour factor \( q = 6 \)
- Ground acceleration \( a = 0.15 \)
- Subsoil class \( g \) type C
  - \( s = 0.8 \)
  - \( T_1 = 0.3 \, s \)
  - \( T_2 = 0.8 \, s \)
  - \( k = 0.667 \)
  - \( \beta_0 = 2.5 \)
- Load combination coefficient \( \psi_{2i} = 0.6 \)

When performing a seismic analysis, if the following condition regarding the natural vibration period is fulfilled:

\[ T_0 \leq 2 \cdot T_2 \]
it is possible to represent the seismic actions with a set of storey horizontal loads, performing a simplified dynamic analysis. These forces, for generic storey (i), are given by:

\[ F_i = \beta(T_0) \cdot z_i \cdot W_i \cdot \sum_{j=1}^{n} W_j \cdot z_j \]

in which:

\[ \beta(T_0) = \frac{a}{g} \cdot \frac{s}{(1 + \frac{T_0}{T_1} \cdot \left( \frac{\eta \cdot \beta_0}{q} - 1 \right))} \quad 0 < T_0 < T_1 \]

\[ \frac{\left( \frac{a}{g} \cdot \eta \cdot s \cdot \beta_0 \right)}{q} \quad T_1 < T_0 < T_2 \]

\[ \frac{\left( \frac{a}{g} \cdot \eta \cdot s \cdot \beta_0 \right)}{q} \cdot \left( \frac{T_2}{T_0} \right)^4 \quad T_2 < T_0 \]

In any case:

\[ \beta(T_0) \geq 0.20 \cdot \frac{a}{g} \]

\[ W_j = W_{p,j} + W_{v,j} \cdot \psi_{z,i} \cdot \phi_j \]

in which:
- \( W_{p,j} \) = storey(j) permanent loads
- \( W_{v,j} \) = storey(j) variable loads
- \( \phi_j \) = storey(j) load factor

In our case:

\[ 1.2 < 2 \cdot 0.8 = 1.6 \]

it is possible to perform a simplified dynamic analysis

\[ \phi_j = 1 \quad \text{For all the five storeys} \]

\[ \eta = 1 \]

These forces have been calculated with the Lotus 1-2-3 spreadsheet given in table 2, that performs a simple execution of the above written formulas. These actions, divided by two, were then applied to the two end nodes of each level.
III - LOAD CASES

The following load case rules have been used:

EC1 - part 1  - For load cases **without seismic loads**

**clause 9.4.5 A) Ultimate limit states (ULS)**

\[ \sum_j \gamma_{G,j} \cdot G_{k,j} + 1.35 \cdot \sum_i Q_{k,i} \]

**clause 9.5.5 B) Serviceability limit states (SLS)**

\[ \sum_j G_{k,j} + 0.9 \cdot \sum_i Q_{k,i} \]

- For load cases **with seismic loads**

EC8 - part 1  **A) Ultimate limit states (ULS)**

**clause 6.2**

\[ \gamma \cdot E + \sum_j G_{k,j} + \sum_i \Psi_{2,i} \cdot Q_{k,i} \]

where:

- \( G_{k,j} \) = characteristic value of the permanent actions
- \( Q_{k,i} \) = characteristic value of the variable actions
\[ \gamma \] = partial factors for permanent actions
\[ \gamma \] = seismic actions importance factor (=1 in our case)
\[ \Psi_{2i} \] = load combination coefficient

**B) Serviceability limit states (SLS)**

The same combination rule used for the ULS was used also for serviceability checks; as it will be more clear further on, appropriate factors are introduced by the EC8 to differentiate the entity of the actions for the two limit states.

The loads considered in this calculation example are:

1) **Permanent actions**
   - SWL
   - DL
   - FIL

2) **Variable actions**
   - IL
   - SL
   - WL

3) **Accidental actions**
   - EL

It is important to notice that the equivalent frame imperfections loads have been considered as "permanent" actions since these characterise the geometrical imperfections of the frame elements and thus are a permanent characteristic of the structure.

It is also important to remember that the self weight loads (SWL) have already been increased by 10%.

The **ULS load cases** that have been considered in this calculation example are:

1) \[ 0.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + 1.35 \cdot FIL \]
2) \[ 0.35 \cdot SWL + 1.35 \cdot DL + 1.35 \cdot IL + 1.35 \cdot FIL + 1.35 \cdot WL \]
3) \[ 0.00 \cdot SWL + 1.00 \cdot DL + 0.6 \cdot IL + 1.00 \cdot FIL + 1.00 \cdot EL \]
The *SLS load cases* that have been considered are:

4) \(0.00 \cdot SWL + 1.00 \cdot DL + 0.9 \cdot IL + 1.00 \cdot FIL + 0.9 \cdot WL\)

**IV - PRELIMINARY SIZING OF MEMBERS**

The predimensioning of the frame beams and columns has been performed with simple preliminary calculations. The chosen profiles are shown in fig.3.

**V - SLS CHECKS**

A preliminary first order analysis of the frame has been carried out. In fig.4 are given the node and member numbers. The SLS verifications that have been
performed regard horizontal displacement checks of the different storeys of the frame.
The calculations methodology and the used formulas differ from each other for load cases with seismic actions to those without seismic actions.

**A) Load cases with seismic loads**

According to the Eurocode 8, the horizontal deflection limits for the relative storey displacements "dr,i" are:

\[
d_{rd} \leq v \cdot 0.006 \cdot h
\]

where:

- \( v \) = reduction factor that takes into account the difference in the loads entity between an "ordinary (serviceability)" earthquake and an "exceptional (ultimate)" one.
- \( h \) = storey height

This displacement can be calculated, for storey(i), by:

\[
d_{ri} = q \cdot d_{res,i} + d_{res,i}
\]

where:

- \( q \) = behaviour factor
- \( d_{res,i} \) = design horizontal displacement (for storey(i)), due to the seismic forces, obtained from a first order analysis
- \( d_{hor,i} \) = design horizontal displacement (for storey(i)), due to the other loads (vertical and horizontal), obtained from a first order analysis

These horizontal storey displacements have been obtained calculating the mean value between the deflections of the two extreme nodes of each storey.

In our case, for \( v = 2.5 \)

\[
q = 6
\]

**for load case 3):**

- **for storey I:**

\[
d_{rd} = 6 \cdot \frac{0.868 + 0.868}{2} + \frac{0.0436 + 0.0804}{2} = 5.270 \times 0.006 \cdot 2.5 \cdot 370 = 5.55 \text{ cm}
\]

- **for storey II:**
\[ d_{r,II} = 6 \cdot \frac{0.412 + 0.412}{2} + \frac{0.0412 + 0.0126}{2} = 2.499 < 0.006 \cdot 2.5 \cdot 310 = 4.65 \text{cm} \]

- for storey III:
\[ d_{r,III} = 6 \cdot \frac{0.290 + 0.290}{2} + \frac{0.0222 + 0.0120}{2} = 1.757 < 0.006 \cdot 2.5 \cdot 310 = 4.65 \text{cm} \]

- for storey IV:
\[ d_{r,IV} = 6 \cdot \frac{0.230 + 0.230}{2} + \frac{0.006 + 0.017}{2} = 1.392 < 0.006 \cdot 2.5 \cdot 310 = 4.65 \text{cm} \]

- for storey V:
\[ d_{r,V} = 6 \cdot \frac{0.140 + 0.140}{2} + \frac{0.049 - 0.035}{2} = 0.847 < 0.006 \cdot 2.5 \cdot 310 = 4.65 \text{cm} \]

B) Load cases without seismic loads

According to the EC3, the horizontal deflection limits for multi-storey buildings are:

EC3
- in each storey:
  clause 4.2.2 (4)
  \[ d_i \leq \frac{h_i}{300} \]
- for the structure as a whole:
  \[ d_{tot} \leq \frac{h}{500} \]

where:
- \( d_{tot} \) = total frame horizontal deflection
- \( d_i \) = storey(i) relative horizontal deflection
- \( h \) = frame total height
- \( h_i \) = storey(i) height

The storeys horizontal deflections were obtained, as it has been done for the cases with seismic loads, by calculating the average value between the horizontal displacements of the two end nodes of each storey. It is also necessary to add that the verification of the storey relative horizontal displacement has only been performed in the first storey since it is less stiff than the second level and thus enables greater displacements.

In our case:

for load case 4):
EC3

section 5.2

A) CHECK FOR "SWAY -NON SWAY" BEHAVIOUR

Before the internal forces and moments are calculated and used in member and cross-sectional checks, it is necessary to check if the non linear geometrical effects (second order) have to be taken into account or not.

A frame is classified as "sway" for a given load condition if, for at least one of the storeys, the following condition is satisfied:

\[
\left( \frac{\delta_i}{h_i} \right) \left( \frac{\sum V_i}{\sum H_i} \right) > 0.1
\]

where:

- \( h_i \) = storey(i) height
- \( \delta_i \) = relative horizontal displacement of storey(i)
- \( \sum V_i \) = total vertical reaction at bottom of storey(i) from first order analysis
- \( \sum H_i \) = total horizontal reaction at the bottom of storey(i) from first order analysis
The relative storey displacements \( \delta_i \) have been obtained with different methods:

1) Load cases with seismic loads

These are calculated in the same way as for the SLS checks:

\[
\delta_{ri} = q \cdot d_{sis,i} + d_{res,i}
\]

2) Load cases without seismic loads

These are taken directly from the first order analysis of the frame.

The values of the "sway-non sway" ratio for the ULS load cases (1, 2 and 3) of this specific example are given in the Lotus 1-2-3- spreadsheets given in tables 3, 4 and 5.

The frame results to be "sway" for load case 3; in this load case it is necessary to take into account the non linear geometrical effects performing a second order analysis. For load cases 1 and 2, instead, it is sufficient to take directly the first order internal forces and moments. It is interesting to observe that these are the non seismic load cases; it is thus evident that being the entity of the seismic actions much greater than that of the other loads, the frame elements are dimensioned mainly on the earthquake effects and the structure results to be over dimensioned for the other actions.

section 5.2.4.2(4) **B) CHECK FOR MEMBER IMPERFECTIONS**
According to the EC3, for load cases in which the frame results to be "sway", it is necessary to check whether the effects of the member imperfections can be neglected or not when performing the global analysis of a structure.

For this purpose, if in a compression member with moment resisting connections the given formula is not fulfilled then it is necessary to take into account these imperfections in the global analysis.

\[
\bar{\lambda} \leq 0.5 \cdot \sqrt{\frac{A \cdot f_y}{N_{sd}}}
\]

where:
- \(\bar{\lambda}\) = in plane non dimensional slenderness calculated using a buckling length equal to the system length
- \(A\) = column cross sectional area
- \(N_{sd}\) = design value of the compression force
- \(f_y\) = design column yield strength

In the scope of this research project, the above written condition should be checked for all the columns of the frame, taking the internal forces and moments of load case 3.

As an example, we can take element 26 (HEB 400) with the internal forces and moments of load case 3.

In our case:
- \(A\) = 198 cm²
- \(N_{sd}\) = 614 KN
- \(f_y\) = 355 N/mm²

\[
\bar{\lambda} = \frac{\bar{\lambda}}{93.9 \cdot \beta} = \frac{370}{93.9 \cdot 0.814} = 1.77
\]

and thus:
\[
\bar{\lambda} = 0.283 < 0.5 \cdot \sqrt{\frac{198 \cdot 355 \cdot 10^{-1}}{614}} = 1.692
\]

If the above written formula is also fulfilled by the other columns, then it is not necessary to take into account the member imperfections in the global analysis of the frame.
C) SECOND ORDER ANALYSIS

The EC3 enables to adopt the rigorous second order calculation method together
with other indirect second order analysis methods.

5.2.6.2 (2)  
1) Rigorous second order analysis method

The direct second order analysis has been performed with "PEP micro". The
results of this calculation (internal forces and moments) can be used directly
for the local member and cross sectional checks that are performed using the
"non sway" buckling lengths.

5.2.6.2 (8)  
2) Sway mode buckling length method
This is one of the indirect second order analysis methods indicated by the EC3; for column member and cross-sectional checks we take directly the first order analysis results and adopt "sway mode" buckling lengths; for the beams it is necessary to increase by 20% the "sway moments"; the beams local member and cross-sectional checks are then performed with the increased values of these moments.

5.2.6.2 (3)

3) Amplified sway moments method

This is the other indirect second order calculation method that is applicable only if, for a given load case, all the storeys of the frame satisfy the following equation:

\[
\left( \frac{\delta_i}{h_i} \right) \left( \frac{\sum V_i}{\sum H_i} \right) \leq 0.25
\]

clause 5.2.6.2(4)

The "sway moments" that result from the first order analysis have to be increased to take into account the additional effects due to the "sway" behaviour of the frame.

For each different storey, the value of the amplification load factor \( \alpha_i \) is obtained with the formula:

\[
\alpha_i = \frac{1}{1 - \left( \frac{\delta_i}{h_i} \right) \left( \frac{\sum V_i}{\sum H_i} \right)}
\]

In our case:
- for load case 3)

LEVEL I
\[
\alpha_I = \frac{1}{1-0.3146} = 1.459
\]

LEVEL II
\[
\alpha_{II} = \frac{1}{1-0.1549} = 1.183
\]

LEVEL III
\[
\alpha_{III} = \frac{1}{1-0.0967} = 1.107
\]

LEVEL IV
\[
\alpha_{IV} = \frac{1}{1-0.0686} = 1.074
\]

LEVEL V
\[
\alpha_{V} = \frac{1}{1-0.0379} = 1.039
\]

It is necessary to observe that, since the value of the "sway-non sway" ratio is above 0.25, for load case 3) this method should not be adopted.; nevertheless, for the scope of this work, it is interesting to have also the results of a "moment amplification analysis".

In this calculation example the following two methods of applying "\(\alpha_i\)" have been adopted:

5.2.6.2(3) a) As it is stated in the EC3, the amplification factors calculated above are used to multiply directly the "sway moments" (that are those moments that result from the horizontal loads and also the vertical loads if either the structure or the loading is asymmetrical). In this way, we obtained incremented values for the bending moments to be used in the local members and cross sectional checks.

This methodology, in the case of symmetrical frames with symmetrical vertical loading, is very simple to adopt (it is in fact sufficient to multiply the part of the moments due to the horizontal loads by the corresponding storey values of "\(\alpha_i\)" and gives results that are sufficiently correct.

In fact, the I order moments of storey(i) "\(M_{i,j}\)" can be expressed in general by:

\[
M_{i,j} = M_i(H) + M_i(V)
\]
where:
H = horizontal loads
V = vertical loads

In the case of symmetrical frames, the second order amplified moments of storey(i) \( M_{n.i} \) are calculated by:

\[
M_{n.j} = \alpha_i \cdot M_i(H) + M_i(V)
\]

In our case the horizontal forces are FIL and EL.

b) Some additional calculations have been performed adopting the modified approach described hereafter: instead of multiplying the "sway moments", factors \( \alpha_i \) were used to multiply the loads that give the horizontal translation of the storeys (i.e. those loads that give the "sway moments").

In our case, according to the definition of "sway moments", it is necessary to multiply only the horizontal loads (i.e. FIL and EL) of load case 3) since the frame is symmetric.

As a consequence, for each different storey of the frame we obtain the following case with increased partial load factors:

- load case 3)

LEVEL I

\[
100 \cdot SWL + 100 \cdot DL + 0.6 \cdot IL + (1.459 \cdot 1.00) \cdot FIL + (1.459 \cdot 1.00) \cdot EL
\]

LEVEL II

\[
100 \cdot SWL + 100 \cdot DL + 0.6 \cdot IL + (1.183 \cdot 1.00) \cdot FIL + (1.183 \cdot 1.00) \cdot EL
\]

LEVEL III

\[
100 \cdot SWL + 100 \cdot DL + 0.6 \cdot IL + (1.107 \cdot 1.00) \cdot FIL + (1.107 \cdot 1.00) \cdot EL
\]

LEVEL IV

\[
100 \cdot SWL + 100 \cdot DL + 0.6 \cdot IL + (1.074 \cdot 1.00) \cdot FIL + (1.074 \cdot 1.00) \cdot EL
\]

LEVEL V

\[
100 \cdot SWL + 100 \cdot DL + 0.6 \cdot IL + (1.039 \cdot 1.00) \cdot FIL + (1.039 \cdot 1.00) \cdot EL
\]
In the case of symmetrical frames, the second order amplified moments of storey(i) "$M_{u,i}$" are calculated by:

$$M_{u,i} = \alpha_i \cdot M_i(H) + M_i(V)$$

In our case the horizontal forces are FIL and EL.

b) Some additional calculations have been performed adopting the modified approach described hereafter: instead of multiplying the "sway moments", factors "$\alpha_i$" were used to multiply the loads that give the horizontal translation of the storeys (i.e. those loads that give the "sway moments"). In our case, according to the definition of "sway moments", it is necessary to multiply only the horizontal loads (i.e. FIL and EL) of load case 3) since the frame is symmetric.

As a consequence, for each different storey of the frame we obtain the following case with increased partial load factors:

- load case 3)

**LEVEL I**

$$1.00 \cdot SWL + 1.00 \cdot DL + 0.6 \cdot IL + (1.459 \cdot 1.00) \cdot FIL + (1.459 \cdot 1.00) \cdot EL$$

**LEVEL II**

$$1.00 \cdot SWL + 1.00 \cdot DL + 0.6 \cdot IL + (1.183 \cdot 1.00) \cdot FIL + (1.183 \cdot 1.00) \cdot EL$$

**LEVEL III**

$$1.00 \cdot SWL + 1.00 \cdot DL + 0.6 \cdot IL + (1.107 \cdot 1.00) \cdot FIL + (1.107 \cdot 1.00) \cdot EL$$

**LEVEL IV**

$$1.00 \cdot SWL + 1.00 \cdot DL + 0.6 \cdot IL + (1.074 \cdot 1.00) \cdot FIL + (1.074 \cdot 1.00) \cdot EL$$

**LEVEL V**

$$1.00 \cdot SWL + 1.00 \cdot DL + 0.6 \cdot IL + (1.039 \cdot 1.00) \cdot FIL + (1.039 \cdot 1.00) \cdot EL$$

With these new load factors for load case 3) a new first order analysis was performed. The results of this analysis are used directly in the local member and cross-sectional verifications in which we adopt the "non sway mode" buckling lengths.
With tables 6 and 7 it is possible to compare the internal forces of:
- Rigorous second order method
- Moment amplification method (with $\alpha_i$ applied to the "sway moments");
- Moment amplification method (with $\alpha_i$ applied to the loads that give the "sway moments").
- First order method for COLUMNS

- Rigorous second order method
- Moment amplification method (with $\alpha_i$ applied to the "sway moments");
- Moment amplification method (with $\alpha_i$ applied to the loads that give the "sway moments").
- Sway mode buckling lengths method
- First order method for BEAMS

The comparison between the two indirect second order methods for the columns is only possible comparing the utilisation factors (expressed as a percentage) that are obtained from the bending-axial force buckling member checks.

The following remarks can be made:

columns - if we examine the values of the bending moments, it is visible that those that are calculated with a rigorous II order analysis are very similar to those of the I order calculation. In addition, the two "moment amplification" calculations seem to give higher moments than the II order analysis; as a consequence, these two methodologies seem to be conservative in this case. Finally, if the two amplification methods are compared, it is visible that if we increment directly the "sway moments" (i.e. adopting the method proposed in the EC3) we obtain larger moments than if we adopt the MAM-F method.
Note: the dimensions are indicated in millimetres
Multi-storey parking (PAR) analysis
Parking areas and circulation ways layout

Note: the dimensions are indicated in metres

ECSC Project 7210-SA/419
Practical application of EUROCODE 3
to multi-storey buildings with steel
"Sway Frame" structures
### FIGURE 4

**Node numbers**

<table>
<thead>
<tr>
<th>6</th>
<th>12</th>
<th>18</th>
<th>24</th>
<th>30</th>
<th>36</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>11</td>
<td>17</td>
<td>23</td>
<td>29</td>
<td>35</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
<td>16</td>
<td>22</td>
<td>28</td>
<td>34</td>
</tr>
<tr>
<td>3</td>
<td>9</td>
<td>15</td>
<td>21</td>
<td>27</td>
<td>33</td>
</tr>
<tr>
<td>2</td>
<td>8</td>
<td>14</td>
<td>20</td>
<td>26</td>
<td>32</td>
</tr>
</tbody>
</table>

**Element numbers**

<table>
<thead>
<tr>
<th>51</th>
<th>52</th>
<th>53</th>
<th>54</th>
<th>55</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>10</td>
<td>15</td>
<td>20</td>
<td>25</td>
</tr>
<tr>
<td>4</td>
<td>11</td>
<td>14</td>
<td>19</td>
<td>24</td>
</tr>
<tr>
<td>3</td>
<td>11</td>
<td>13</td>
<td>18</td>
<td>23</td>
</tr>
<tr>
<td>2</td>
<td>12</td>
<td>13</td>
<td>17</td>
<td>22</td>
</tr>
<tr>
<td>1</td>
<td>16</td>
<td>11</td>
<td>16</td>
<td>21</td>
</tr>
</tbody>
</table>
**ECSC Project SA/419 - "EC3 Sway Frames"**  
**EUROCODE 1: WIND ACTIONS**

**Title:** Multi-storey parking building (PAR) calculation example  
**Date:**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_t$ topography coefficient</td>
<td>1</td>
</tr>
<tr>
<td>$D$ building diagonal length</td>
<td>62.1 m</td>
</tr>
<tr>
<td>$C_{DIR}$ direction factor</td>
<td>1</td>
</tr>
<tr>
<td>$C_{TEM}$ seasonal factor</td>
<td>1</td>
</tr>
<tr>
<td>$a_s$ height above sea level</td>
<td>0 (Km)</td>
</tr>
<tr>
<td>$K_0$ $a_s$ coefficient</td>
<td>0.55 (1/Km)</td>
</tr>
<tr>
<td>$V_{ref,0}$ reference wind speed for $a_s=0$</td>
<td>28 m/s</td>
</tr>
<tr>
<td>$B$ distance between transversal frames</td>
<td>7.5 m</td>
</tr>
<tr>
<td>$K_r$ roughness factor</td>
<td>0.24</td>
</tr>
<tr>
<td>$z_0$ roughness length</td>
<td>1 (m)</td>
</tr>
<tr>
<td>$z_{min}$ minimum height</td>
<td>16 (m)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Level</th>
<th>building portion m</th>
<th>reference elevation m</th>
<th>positive pressure coefficient</th>
<th>negative pressure coefficient</th>
<th>positive storey load KN</th>
<th>negative storey load KN</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>3.4</td>
<td>3.7</td>
<td>0.8</td>
<td>0.3</td>
<td>10.52</td>
<td>3.95</td>
</tr>
<tr>
<td>II</td>
<td>3.1</td>
<td>6.8</td>
<td>0.8</td>
<td>0.3</td>
<td>9.59</td>
<td>3.60</td>
</tr>
<tr>
<td>III</td>
<td>3.1</td>
<td>9.9</td>
<td>0.8</td>
<td>0.3</td>
<td>9.59</td>
<td>3.60</td>
</tr>
<tr>
<td>IV</td>
<td>3.1</td>
<td>13.0</td>
<td>0.8</td>
<td>0.3</td>
<td>9.59</td>
<td>3.60</td>
</tr>
<tr>
<td>V</td>
<td>1.55</td>
<td>16.1</td>
<td>0.8</td>
<td>0.3</td>
<td>4.81</td>
<td>1.80</td>
</tr>
<tr>
<td>VI</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>VII</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>VIII</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>IX</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**TABLE 1**
**ECSC Project SA/419 - “EC3 Sway Frames”**

**EUROCODE 8: SEISMIC ACTIONS**

<table>
<thead>
<tr>
<th>( X ) (structural damping)</th>
<th>0.5 %</th>
<th>( T ) (frame natural period)</th>
<th>1.2 (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a/g ) (ground acceleration)</td>
<td>0.15</td>
<td>( \eta ) (damping correction factor)</td>
<td>1</td>
</tr>
<tr>
<td>( s ) (foundation coefficient)</td>
<td>0.8</td>
<td>( T_1 )</td>
<td>0.3 (s)</td>
</tr>
<tr>
<td>( q ) (structure coefficient)</td>
<td>6</td>
<td>( T_2 )</td>
<td>0.8 (s)</td>
</tr>
<tr>
<td>( \Psi_{2i} ) (load combination coeff.)</td>
<td>0.6</td>
<td>( k )</td>
<td>0.6667</td>
</tr>
</tbody>
</table>

**Wpi** = dead loads of storey \( i \)

**Wvi** = imposed loads of storey \( i \)

**Wtot \(_i\)** = total storey \( i \) loads (with reduced imposed loads)

**Fi** = storey \( i \) seismic loads

**Zi** = storey \( i \) level height

**\( \psi_{i} \)** = storey \( i \) reduction load factor

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>Wpi</th>
<th>Wvi</th>
<th>( \psi_{i} )</th>
<th>Wtot (_i)</th>
<th>Zi</th>
<th>Fi</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1440</td>
<td>480</td>
<td>1</td>
<td>1728</td>
<td>3.7</td>
<td>24.64</td>
</tr>
<tr>
<td>II</td>
<td>1440</td>
<td>480</td>
<td>1</td>
<td>1728</td>
<td>6.8</td>
<td>45.29</td>
</tr>
<tr>
<td>III</td>
<td>1440</td>
<td>480</td>
<td>1</td>
<td>1728</td>
<td>9.9</td>
<td>65.93</td>
</tr>
<tr>
<td>IV</td>
<td>1440</td>
<td>480</td>
<td>1</td>
<td>1728</td>
<td>13</td>
<td>86.58</td>
</tr>
<tr>
<td>V</td>
<td>1440</td>
<td>480</td>
<td>1</td>
<td>1728</td>
<td>16.1</td>
<td>107.23</td>
</tr>
<tr>
<td>VI</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.00</td>
</tr>
<tr>
<td>VII</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.00</td>
</tr>
<tr>
<td>VIII</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**TABLE 2**
### Table 3: EC3 Sway Frames - EUROCODE 3: SWAY - NON SWAY CRITERION

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (cm)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
<td>729</td>
<td>6,880</td>
<td>0.000</td>
<td>2</td>
<td>0.056</td>
<td>0.056</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>2,190</td>
<td>24,700</td>
<td></td>
<td>8</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>13</td>
<td>3,000</td>
<td>-7,420</td>
<td></td>
<td>14</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>19</td>
<td>3,000</td>
<td>-6,070</td>
<td></td>
<td>20</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>25</td>
<td>2,190</td>
<td>-7,900</td>
<td></td>
<td>26</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>31</td>
<td>749</td>
<td>-1,800</td>
<td>0.000</td>
<td>32</td>
<td>0.112</td>
<td>0.112</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>11,858</td>
<td>-32,710</td>
<td>Storey height (cm)</td>
<td>370</td>
<td>mean rel. (cm)</td>
<td>0.084</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: \((\frac{N \text{ tot } x \text{ st. disp.}}{h \times V \text{ tot}})\) = 0.0822

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (cm)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1</td>
<td>2</td>
<td>590</td>
<td>27,400</td>
<td>0.056</td>
<td>3</td>
<td>0.114</td>
<td>0.058</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>1,730</td>
<td>73,500</td>
<td></td>
<td>9</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>14</td>
<td>2,400</td>
<td>-14,500</td>
<td></td>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>20</td>
<td>2,400</td>
<td>0.657</td>
<td></td>
<td>21</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>26</td>
<td>1,730</td>
<td>-82,700</td>
<td></td>
<td>27</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>32</td>
<td>601</td>
<td>-30,500</td>
<td>0.112</td>
<td>33</td>
<td>0.126</td>
<td>0.014</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>9,451</td>
<td>-26,143</td>
<td>Storey height (cm)</td>
<td>310</td>
<td>mean rel. (cm)</td>
<td>0.036</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: \((\frac{N \text{ tot } x \text{ st. disp.}}{h \times V \text{ tot}})\) = 0.0421

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (cm)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>1</td>
<td>3</td>
<td>443</td>
<td>32,500</td>
<td>0.114</td>
<td>4</td>
<td>0.144</td>
<td>0.030</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>1,300</td>
<td>83,300</td>
<td></td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>15</td>
<td>1,800</td>
<td>-10,700</td>
<td></td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>21</td>
<td>1,800</td>
<td>0.779</td>
<td></td>
<td>22</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>27</td>
<td>1,300</td>
<td>-90,700</td>
<td></td>
<td>28</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>33</td>
<td>449</td>
<td>-34,700</td>
<td>0.126</td>
<td>34</td>
<td>0.142</td>
<td>0.016</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>7,092</td>
<td>-19,521</td>
<td>Storey height (cm)</td>
<td>310</td>
<td>mean rel. (cm)</td>
<td>0.023</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: \((\frac{N \text{ tot } x \text{ st. disp.}}{h \times V \text{ tot}})\) = 0.0270

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (cm)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>1</td>
<td>4</td>
<td>291</td>
<td>34,800</td>
<td>0.144</td>
<td>5</td>
<td>0.152</td>
<td>0.008</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10</td>
<td>866</td>
<td>76,100</td>
<td></td>
<td>11</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>1,200</td>
<td>-4,000</td>
<td></td>
<td>17</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>22</td>
<td>1,200</td>
<td>-2,410</td>
<td></td>
<td>23</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>28</td>
<td>886</td>
<td>-81,000</td>
<td></td>
<td>29</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>34</td>
<td>293</td>
<td>36,600</td>
<td>0.142</td>
<td>35</td>
<td>0.166</td>
<td>0.024</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>4,716</td>
<td>-13,110</td>
<td>Storey height (cm)</td>
<td>310</td>
<td>mean rel. (cm)</td>
<td>0.016</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: \((\frac{N \text{ tot } x \text{ st. disp.}}{h \times V \text{ tot}})\) = 0.0186

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (cm)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>1</td>
<td>5</td>
<td>137</td>
<td>39,600</td>
<td>0.152</td>
<td>6</td>
<td>0.225</td>
<td>0.073</td>
</tr>
<tr>
<td></td>
<td></td>
<td>11</td>
<td>437</td>
<td>94,800</td>
<td></td>
<td>12</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>17</td>
<td>605</td>
<td>-19,300</td>
<td></td>
<td>18</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>23</td>
<td>605</td>
<td>15,800</td>
<td></td>
<td>24</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>29</td>
<td>437</td>
<td>-97,400</td>
<td></td>
<td>30</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>35</td>
<td>138</td>
<td>-40,100</td>
<td>0.166</td>
<td>36</td>
<td>0.111</td>
<td>0.055</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>2,359</td>
<td>-6,600</td>
<td>Storey height (cm)</td>
<td>310</td>
<td>mean rel. (cm)</td>
<td>0.009</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: \((\frac{N \text{ tot } x \text{ st. disp.}}{h \times V \text{ tot}})\) = 0.0104
<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>Total disp. (cm)</th>
<th>Upper nodes</th>
<th>Total disp. (cm)</th>
<th>Relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>1</td>
<td>706</td>
<td>0.597</td>
<td>0.000</td>
<td>2</td>
<td>0.266</td>
<td>0.266</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>2190</td>
<td>9.200</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>13</td>
<td>3000</td>
<td>-26.900</td>
<td>14</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>19</td>
<td>3000</td>
<td>-27.400</td>
<td>20</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>25</td>
<td>2190</td>
<td>-32.300</td>
<td>26</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>31</td>
<td>773</td>
<td>-17.800</td>
<td>0.000</td>
<td>32</td>
<td>0.317</td>
<td>0.317</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>11859</td>
<td>-114.603</td>
<td>Storey height</td>
<td>370</td>
<td>mean rel.</td>
<td>0.292</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: \( \frac{\text{N tot} \times \text{u disp}}{h \times \text{V tot}} \) = 0.0813

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>Total disp. (cm)</th>
<th>Upper nodes</th>
<th>Total disp. (cm)</th>
<th>Relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>2</td>
<td>2</td>
<td>578</td>
<td>23.800</td>
<td>0.266</td>
<td>3</td>
<td>0.472</td>
<td>0.146</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>1730</td>
<td>62.600</td>
<td>9</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>14</td>
<td>2400</td>
<td>-31.000</td>
<td>15</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>20</td>
<td>2400</td>
<td>-15.900</td>
<td>21</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>26</td>
<td>1730</td>
<td>-93.700</td>
<td>27</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>32</td>
<td>613</td>
<td>-34.300</td>
<td>0.317</td>
<td>33</td>
<td>0.419</td>
<td>0.102</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>9451</td>
<td>-88.500</td>
<td>Storey height</td>
<td>310</td>
<td>mean rel.</td>
<td>0.124</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: \( \frac{\text{N tot} \times \text{u disp}}{h \times \text{V tot}} \) = 0.0427

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>Total disp. (cm)</th>
<th>Upper nodes</th>
<th>Total disp. (cm)</th>
<th>Relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>2</td>
<td>3</td>
<td>437</td>
<td>30.100</td>
<td>0.412</td>
<td>4</td>
<td>0.495</td>
<td>0.083</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>1300</td>
<td>74.700</td>
<td>10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>15</td>
<td>1800</td>
<td>-22.000</td>
<td>16</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>21</td>
<td>1800</td>
<td>-10.500</td>
<td>22</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>27</td>
<td>1300</td>
<td>-99.300</td>
<td>28</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>33</td>
<td>455</td>
<td>-37.200</td>
<td>0.317</td>
<td>34</td>
<td>0.488</td>
<td>0.069</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>7092</td>
<td>-68.300</td>
<td>Storey height</td>
<td>310</td>
<td>mean rel.</td>
<td>0.076</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: \( \frac{\text{N tot} \times \text{u disp}}{h \times \text{V tot}} \) = 0.0271

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>Total disp. (cm)</th>
<th>Upper nodes</th>
<th>Total disp. (cm)</th>
<th>Relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>2</td>
<td>4</td>
<td>288</td>
<td>32.800</td>
<td>0.495</td>
<td>5</td>
<td>0.537</td>
<td>0.042</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10</td>
<td>866</td>
<td>71.200</td>
<td>11</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>1200</td>
<td>-10.500</td>
<td>17</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>22</td>
<td>1200</td>
<td>-8.910</td>
<td>23</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>28</td>
<td>847</td>
<td>-8.500</td>
<td>29</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>34</td>
<td>296</td>
<td>-38.500</td>
<td>0.488</td>
<td>35</td>
<td>0.546</td>
<td>0.058</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>4717</td>
<td>-39.810</td>
<td>Storey height</td>
<td>310</td>
<td>mean rel.</td>
<td>0.050</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: \( \frac{\text{N tot} \times \text{u disp}}{h \times \text{V tot}} \) = 0.0191

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>Total disp. (cm)</th>
<th>Upper nodes</th>
<th>Total disp. (cm)</th>
<th>Relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>2</td>
<td>5</td>
<td>137</td>
<td>39.600</td>
<td>0.537</td>
<td>6</td>
<td>0.622</td>
<td>0.085</td>
</tr>
<tr>
<td></td>
<td></td>
<td>11</td>
<td>437</td>
<td>93.200</td>
<td>12</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>17</td>
<td>605</td>
<td>-21.900</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>23</td>
<td>605</td>
<td>13.100</td>
<td>24</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>29</td>
<td>438</td>
<td>-99.200</td>
<td>30</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>35</td>
<td>139</td>
<td>-40.300</td>
<td>0.546</td>
<td>36</td>
<td>0.506</td>
<td>-0.040</td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>2361</td>
<td>-15.500</td>
<td>Storey height</td>
<td>310</td>
<td>mean rel.</td>
<td>0.022</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: \( \frac{\text{N tot} \times \text{u disp}}{h \times \text{V tot}} \) = 0.0111

**TABLE 4**
<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (KN)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>1</td>
<td>357</td>
<td>19.800</td>
<td>0.000</td>
<td>2</td>
<td>5.252</td>
<td>5.252</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>1440</td>
<td>-46.400</td>
<td>8</td>
<td>14</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>13</td>
<td>1980</td>
<td>-84.000</td>
<td>14</td>
<td>1</td>
<td>1980</td>
<td>-84.000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>19</td>
<td>1980</td>
<td>-84.500</td>
<td>20</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>25</td>
<td>1440</td>
<td>-87.100</td>
<td>26</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>31</td>
<td>620</td>
<td>-32.100</td>
<td>0.000</td>
<td>31</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>7817</td>
<td>-353.900</td>
<td>Storey height (cm)</td>
<td>370 mean rel.</td>
<td>5.276</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: $\frac{N \text{ tot } \times \text{disp.}}{h \times V \text{ tot}} = 0.3146$

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (KN)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td></td>
<td>2</td>
<td>315</td>
<td>-1.010</td>
<td>5.252</td>
<td>3</td>
<td>7.765</td>
<td>2.513</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>1140</td>
<td>-6.020</td>
<td>9</td>
<td>14</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>14</td>
<td>1580</td>
<td>-89.700</td>
<td>15</td>
<td>1</td>
<td>1580</td>
<td>-89.700</td>
</tr>
<tr>
<td></td>
<td></td>
<td>20</td>
<td>1590</td>
<td>-79.800</td>
<td>21</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>26</td>
<td>1140</td>
<td>-109.000</td>
<td>27</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>32</td>
<td>472</td>
<td>-39.100</td>
<td>5.288</td>
<td>33</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>6237</td>
<td>-324.630</td>
<td>Storey height (cm)</td>
<td>310 mean rel.</td>
<td>2.499</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: $\frac{N \text{ tot } \times \text{disp.}}{h \times V \text{ tot}} = 0.1549$

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (KN)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td></td>
<td>3</td>
<td>246</td>
<td>6.210</td>
<td>7.765</td>
<td>4</td>
<td>9.527</td>
<td>1.762</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>856</td>
<td>-55.50</td>
<td>10</td>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>15</td>
<td>1190</td>
<td>-73.300</td>
<td>16</td>
<td>1</td>
<td>1190</td>
<td>-73.300</td>
</tr>
<tr>
<td></td>
<td></td>
<td>21</td>
<td>1190</td>
<td>-65.700</td>
<td>22</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>27</td>
<td>856</td>
<td>-109.000</td>
<td>28</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>33</td>
<td>341</td>
<td>-38.000</td>
<td>7.772</td>
<td>34</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>4681</td>
<td>-274.240</td>
<td>Storey height (cm)</td>
<td>310 mean rel.</td>
<td>1.757</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: $\frac{N \text{ tot } \times \text{disp.}}{h \times V \text{ tot}} = 0.0967$

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (KN)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td></td>
<td>4</td>
<td>171</td>
<td>8.750</td>
<td>9.527</td>
<td>5</td>
<td>10.913</td>
<td>1.386</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10</td>
<td>571</td>
<td>14.200</td>
<td>11</td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>793</td>
<td>-50.100</td>
<td>17</td>
<td>22</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>22</td>
<td>792</td>
<td>-49.000</td>
<td>23</td>
<td>28</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>28</td>
<td>571</td>
<td>-89.200</td>
<td>29</td>
<td>34</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>34</td>
<td>215</td>
<td>-38.200</td>
<td>9.525</td>
<td>35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>3113</td>
<td>-203.550</td>
<td>Storey height (cm)</td>
<td>310 mean rel.</td>
<td>1.392</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: $\frac{N \text{ tot } \times \text{disp.}}{h \times V \text{ tot}} = 0.0066$

<table>
<thead>
<tr>
<th>Storey</th>
<th>Load Comb.</th>
<th>Lower nodes</th>
<th>N (KN)</th>
<th>V (KN)</th>
<th>total disp. (cm)</th>
<th>Upper nodes</th>
<th>total disp. (KN)</th>
<th>relative disp. (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td>5</td>
<td>84</td>
<td>20.600</td>
<td>10.913</td>
<td>6</td>
<td>11.802</td>
<td>0.889</td>
</tr>
<tr>
<td></td>
<td></td>
<td>11</td>
<td>288</td>
<td>41.900</td>
<td>12</td>
<td>1</td>
<td>288</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>17</td>
<td>399</td>
<td>-40.600</td>
<td>18</td>
<td>3</td>
<td>399</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>23</td>
<td>399</td>
<td>-17.500</td>
<td>24</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>29</td>
<td>288</td>
<td>-84.700</td>
<td>30</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>35</td>
<td>98</td>
<td>-31.800</td>
<td>10.922</td>
<td>36</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storey total forces (KN)</td>
<td>1556</td>
<td>-112.100</td>
<td>Storey height (cm)</td>
<td>310 mean rel.</td>
<td>0.847</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sway criterion: $\frac{N \text{ tot } \times \text{disp.}}{h \times V \text{ tot}} = 0.0027$

**TABLE 5**
### ECSC Project 5A/419 - "EC3 Sway Frames"

### EC3: "SWAY FRAMES" ANALYSIS RESULTS

**Title:** Multi storey parking building (PAR) - Seismic design - COLUMNS

**Date:**

<table>
<thead>
<tr>
<th>Elem.</th>
<th>Node</th>
<th>Comb.</th>
<th>I order</th>
<th>II order - MAM-F</th>
<th>II order - MAM-M</th>
<th>II order - SMBL</th>
<th>II order</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>N [kN]</td>
<td>V [kN]</td>
<td>M [kN*m]</td>
<td>N [kN]</td>
<td>V [kN]</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
<td>352</td>
<td>19.8</td>
<td>73.3</td>
<td>341</td>
<td>23.0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>3</td>
<td>310</td>
<td>1.00</td>
<td>22.6</td>
<td>305</td>
<td>2.53</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>3</td>
<td>244</td>
<td>-6.22</td>
<td>-2.97</td>
<td>241</td>
<td>-5.24</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>3</td>
<td>166</td>
<td>-8.76</td>
<td>-8.00</td>
<td>165</td>
<td>-8.00</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>3</td>
<td>79.4</td>
<td>-20.7</td>
<td>-34.0</td>
<td>79.1</td>
<td>-20.6</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>3</td>
<td>314</td>
<td>32.1</td>
<td>119</td>
<td>625</td>
<td>35.2</td>
</tr>
<tr>
<td></td>
<td>27</td>
<td>3</td>
<td>468</td>
<td>39.1</td>
<td>80.2</td>
<td>473</td>
<td>40.6</td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>3</td>
<td>336</td>
<td>38.0</td>
<td>58.2</td>
<td>339</td>
<td>39.0</td>
</tr>
<tr>
<td></td>
<td>29</td>
<td>3</td>
<td>211</td>
<td>38.2</td>
<td>68.0</td>
<td>212</td>
<td>38.9</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>3</td>
<td>93.4</td>
<td>31.8</td>
<td>93.7</td>
<td>31.9</td>
<td>65.2</td>
</tr>
</tbody>
</table>

**EC3: "SWAY FRAMES" ANALYSIS RESULTS**

**INTERNAL FORCES AND MOMENTS**

**TYPE OF ANALYSIS**

- **I order**
  - First order analysis

- **II order - MAM-F**
  - Indirect second order analysis - Moment amplification method; amplification factors applied to the loads that give the "sway moments"

- **II order - MAM-M**
  - Indirect second order analysis - Moment amplification method; amplification factors applied to "sway moments"

- **II order - SMBL**
  - Indirect second order analysis - Sway mode buckling lengths method

- **II order**
  - Rigorous second order analysis

**TABLE 6**
<table>
<thead>
<tr>
<th>Node</th>
<th>Comb. Num.</th>
<th>Comb. Type</th>
<th>N (kN)</th>
<th>V (kN)</th>
<th>M (kN*m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td>8</td>
<td>MAM-F</td>
<td>137</td>
<td>-29.4</td>
<td>132</td>
</tr>
<tr>
<td>37</td>
<td>9</td>
<td>MAM-M</td>
<td>155</td>
<td>-106</td>
<td>153</td>
</tr>
<tr>
<td>37</td>
<td>15</td>
<td>MAM-M</td>
<td>230</td>
<td>-405</td>
<td>227</td>
</tr>
<tr>
<td>34</td>
<td>20</td>
<td>MAM-M</td>
<td>139</td>
<td>-41.6</td>
<td>145</td>
</tr>
<tr>
<td>34</td>
<td>26</td>
<td>MAM-M</td>
<td>164</td>
<td>-137</td>
<td>164</td>
</tr>
<tr>
<td>39</td>
<td>21</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>171</td>
<td>-164</td>
<td>172</td>
</tr>
<tr>
<td>39</td>
<td>27</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>222</td>
<td>-368</td>
<td>219</td>
</tr>
<tr>
<td>44</td>
<td>22</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>171</td>
<td>-164</td>
<td>172</td>
</tr>
<tr>
<td>44</td>
<td>28</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>212</td>
<td>-330</td>
<td>211</td>
</tr>
<tr>
<td>49</td>
<td>23</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>181</td>
<td>-204</td>
<td>181</td>
</tr>
<tr>
<td>49</td>
<td>29</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>202</td>
<td>-290</td>
<td>201</td>
</tr>
<tr>
<td>54</td>
<td>24</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>194</td>
<td>-249</td>
<td>195</td>
</tr>
<tr>
<td>54</td>
<td>30</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>188</td>
<td>-224</td>
<td>188</td>
</tr>
</tbody>
</table>

**TYPE OF ANALYSIS**

- **I order**: First order analysis
- **II order - MAM-F**: Indirect second order analysis - Moment amplification method; amplification factors applied to the loads that give the "sway moments"
- **II order - MAM-M**: Indirect second order analysis - Moment amplification method; amplification factors applied to "sway moments"
- **II order - Sway Mode Buckling Lengths Method**: Indirect second order analysis - Sway mode buckling lengths method
- **II order**: Rigorous second order analysis

**TABLE 7**

<table>
<thead>
<tr>
<th>Comb.</th>
<th>IV order</th>
<th>V (kN)</th>
<th>M (kN*m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td>104</td>
<td>-505</td>
<td>-80.8</td>
</tr>
<tr>
<td>37</td>
<td>145</td>
<td>22.8</td>
<td>143</td>
</tr>
<tr>
<td>39</td>
<td>164</td>
<td>-110</td>
<td>163</td>
</tr>
<tr>
<td>44</td>
<td>172</td>
<td>-151</td>
<td>172</td>
</tr>
<tr>
<td>49</td>
<td>181</td>
<td>-194</td>
<td>181</td>
</tr>
<tr>
<td>54</td>
<td>195</td>
<td>-244</td>
<td>195</td>
</tr>
</tbody>
</table>

**INTERNAL FORCES AND MOMENTS**

<table>
<thead>
<tr>
<th>Node</th>
<th>Comb. Num.</th>
<th>Comb. Type</th>
<th>N (kN)</th>
<th>V (kN)</th>
<th>M (kN*m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td>8</td>
<td>MAM-F</td>
<td>137</td>
<td>-29.4</td>
<td>132</td>
</tr>
<tr>
<td>37</td>
<td>9</td>
<td>MAM-M</td>
<td>155</td>
<td>-106</td>
<td>153</td>
</tr>
<tr>
<td>37</td>
<td>15</td>
<td>MAM-M</td>
<td>230</td>
<td>-405</td>
<td>227</td>
</tr>
<tr>
<td>34</td>
<td>20</td>
<td>MAM-M</td>
<td>139</td>
<td>-41.6</td>
<td>145</td>
</tr>
<tr>
<td>34</td>
<td>26</td>
<td>MAM-M</td>
<td>164</td>
<td>-137</td>
<td>164</td>
</tr>
<tr>
<td>39</td>
<td>21</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>171</td>
<td>-164</td>
<td>172</td>
</tr>
<tr>
<td>39</td>
<td>27</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>222</td>
<td>-368</td>
<td>219</td>
</tr>
<tr>
<td>44</td>
<td>22</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>171</td>
<td>-164</td>
<td>172</td>
</tr>
<tr>
<td>44</td>
<td>28</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>212</td>
<td>-330</td>
<td>211</td>
</tr>
<tr>
<td>49</td>
<td>23</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>181</td>
<td>-204</td>
<td>181</td>
</tr>
<tr>
<td>49</td>
<td>29</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>202</td>
<td>-290</td>
<td>201</td>
</tr>
<tr>
<td>54</td>
<td>24</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>194</td>
<td>-249</td>
<td>195</td>
</tr>
<tr>
<td>54</td>
<td>30</td>
<td>Sway Mode Buckling Lengths Method</td>
<td>188</td>
<td>-224</td>
<td>188</td>
</tr>
</tbody>
</table>

**INTERNAL FORCES AND MOMENTS**

<table>
<thead>
<tr>
<th>Comb.</th>
<th>IV order</th>
<th>V (kN)</th>
<th>M (kN*m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td>104</td>
<td>-505</td>
<td>-80.8</td>
</tr>
<tr>
<td>37</td>
<td>145</td>
<td>22.8</td>
<td>143</td>
</tr>
<tr>
<td>39</td>
<td>164</td>
<td>-110</td>
<td>163</td>
</tr>
<tr>
<td>44</td>
<td>172</td>
<td>-151</td>
<td>172</td>
</tr>
<tr>
<td>49</td>
<td>181</td>
<td>-194</td>
<td>181</td>
</tr>
<tr>
<td>54</td>
<td>195</td>
<td>-244</td>
<td>195</td>
</tr>
</tbody>
</table>

**INTERNAL FORCES AND MOMENTS**

<table>
<thead>
<tr>
<th>Comb.</th>
<th>IV order</th>
<th>V (kN)</th>
<th>M (kN*m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td>104</td>
<td>-505</td>
<td>-80.8</td>
</tr>
<tr>
<td>37</td>
<td>145</td>
<td>22.8</td>
<td>143</td>
</tr>
<tr>
<td>39</td>
<td>164</td>
<td>-110</td>
<td>163</td>
</tr>
<tr>
<td>44</td>
<td>172</td>
<td>-151</td>
<td>172</td>
</tr>
<tr>
<td>49</td>
<td>181</td>
<td>-194</td>
<td>181</td>
</tr>
<tr>
<td>54</td>
<td>195</td>
<td>-244</td>
<td>195</td>
</tr>
</tbody>
</table>

**INTERNAL FORCES AND MOMENTS**

<table>
<thead>
<tr>
<th>Comb.</th>
<th>IV order</th>
<th>V (kN)</th>
<th>M (kN*m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td>104</td>
<td>-505</td>
<td>-80.8</td>
</tr>
<tr>
<td>37</td>
<td>145</td>
<td>22.8</td>
<td>143</td>
</tr>
<tr>
<td>39</td>
<td>164</td>
<td>-110</td>
<td>163</td>
</tr>
<tr>
<td>44</td>
<td>172</td>
<td>-151</td>
<td>172</td>
</tr>
<tr>
<td>49</td>
<td>181</td>
<td>-194</td>
<td>181</td>
</tr>
<tr>
<td>54</td>
<td>195</td>
<td>-244</td>
<td>195</td>
</tr>
</tbody>
</table>

**INTERNAL FORCES AND MOMENTS**

<table>
<thead>
<tr>
<th>Comb.</th>
<th>IV order</th>
<th>V (kN)</th>
<th>M (kN*m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td>104</td>
<td>-505</td>
<td>-80.8</td>
</tr>
<tr>
<td>37</td>
<td>145</td>
<td>22.8</td>
<td>143</td>
</tr>
<tr>
<td>39</td>
<td>164</td>
<td>-110</td>
<td>163</td>
</tr>
<tr>
<td>44</td>
<td>172</td>
<td>-151</td>
<td>172</td>
</tr>
<tr>
<td>49</td>
<td>181</td>
<td>-194</td>
<td>181</td>
</tr>
<tr>
<td>54</td>
<td>195</td>
<td>-244</td>
<td>195</td>
</tr>
</tbody>
</table>
ANNEX III

The Second Order Computer Programs
1 - Type of program and structures handled by the code
"ABAQUS" is a multi purpose computer code including numerous 2 or 3 dimensional elements. The "ABAQUS/Standard" module is given with the ABAQUS/Post iterative post-processor.

2 - Analysis methods
FEM method; the code can perform ordinary first order analysis and also second order analysis in which it is possible to account for the effects of the nodal displacements in the determination of the internal forces. In addition, "ABAQUS" performs elasto-plastic, rigid-plastic, elastic visco-plastic, creep analysis, etc.

3 - ULS analysis methods for "sway frames" according to the EC3 (section 5.2.6.2)
The first order analysis enables to adopt the "Sway mode buckling lengths" method (SMBL) and the "Moment amplification" method (MAM). The second order analysis enables the "Rigorous second order" method.

4 - Structural model generation - Input data mode
Input is organised around "keywords" and their associated data; many simple mesh generation options are available. All these input lines are scanned and interpreted for consistency in the pre-processor which performs input data checking.

5 - Catalogues of components in the data base
Not implemented for steel sections.

6 - Automatic member checks
Not implemented for steel sections.

7 - Outputs
It is possible to print out the results in tabular form. The user may define which variable appears in each column of each table, thus designing the output for specific needs. In addition, the results of the calculation can be displayed graphically (deflected structure, undeflected shape on deformed shape, internal forces diagrams for a member and for the whole structure, support reactions).
**NAME OF CODE:** Adina  
**COMPANY:** Adina R & D, Inc. - USA

**TYPICAL HARDWARE CONFIGURATION:**  
PC compatible 386 or above (2 Mb RAM) - Workstation  
**OPERATING SYSTEM:** MS-Dos - Unix

**OPERATIONAL FEATURES:**

1. **Type of program and structures handled by the code**  
   "Adina" is a multi purpose computer code including numerous 2 or 3 dimensional elements. The pre processor of the "Adina" system is "Adina-IN"; the post processor is "Adina-PLOT". The code performs the analysis of 2D or 3D models taking some of the many elements available in the library.

2. **Analysis methods**  
   FEM method; the code can perform ordinary first order analysis and also second order analysis in which it is possible to account for the effects of the nodal displacements in the determination of the internal forces. In addition, "Adina" performs elasto-plastic, rigid-plastic, creep analysis, etc together with dynamic analysis.

3. **ULS analysis methods for "sway frames" according to the EC3 (section 5.2.6.2)**  
   The first order analysis enables to adopt the "Sway mode buckling lengths" method (SMBL) and the "Moment amplification" method (MAM).  
   The second order analysis enables the "Rigorous second order" method.

4. **Structural model generation - Input data mode**  
   The commands can be entered at the keyboard, in an interactive mode, or read from a disk file in a non interactive mode. When the input parameters are given with "Adina-IN", a number of mesh geometry plotting features are available: display of node numbers, element numbers, boundary conditions, etc in order to check the structural model.

5. **Catalogues of components in the data base**  
   "Adina" has two additional post-processors for steel beam-column frames; these are "Steelver" and "Module EC3" respectively for the A.I.S.C. 80 and for the Eurocode 3. Both modules have data bases for I, H, channel, sections.

6. **Automatic member checks**  
   "Steelver" and "Module EC3" perform elementary resistance and bucking checks of frame elements (e.g. compression resistance, bending resistance, compression buckling resistance, etc).

7. **Outputs**  
   It is possible to print out the results in tabular form. The user may define which variable appears in each column of each table, thus designing the output for specific needs. In addition, the results of the calculation can be displayed graphically (deflected structure, undeflected shape on deformed shape, internal forces diagrams for a member and for the whole structure, support reactions).
1 - Type of program and structures handled by the code

"ANSYS" is a multi purpose computer code. The code handles any type of 2D or 3D model. The three phases of the finite element analysis (pre processing, solution and post processing) are fully INTEGRATED and thus no additional software package is needed for generating a model or examining the analysis results.

2 - Analysis methods

FEM method; the code can perform ordinary first order analysis and also second order analysis in which it is possible to account for the effects of the nodal displacements in the determination of the internal forces. In addition, "ANSYS" performs dynamic, heat transfer, magnetic, etc analysis and can deal with other types of nonlinearities such as material temperature, etc.

3 - ULS analysis methods for "sway frames" according to the EC3 (section 5.2.6.2)

The first order analysis enables to adopt the "Sway mode buckling lengths" method (SMBL) and the "Moment amplification" method (MAM).

The second order analysis enables the "Rigorous second order" method.

4 - Structural model generation - Input data mode

The input data can be entered by means of a mouse, a keyboard, or a combination of the two. The user interface offers interactive access to commands, control panels, button menus, and reference material through a structured tree-like menu system that includes extensive on line documentation, a command editor, menu pop-ups, data tables and on line help.

5 - Catalogues of components in the data base

Not implemented for steel structures.

6 - Automatic member checks

Not implemented for steel structures.

7 - Outputs

The output from the post processing phase is in graphic display and /or tabular report form. Displays may be made on-line during an interactive post-processing session at a graphics display device or may be diverted for off-line plotting. The results may include displacements, stresses, temperatures, etc.
1 - Type of program and structures handled by the code
   The code is a multi-purpose code for 2D or 3D models. The non linear analysis are
   performed by the "NSTAR" non linear module; for the creation of the structural model
   the pre-post processor "GEOSTAR" is necessary.

2 - Analysis methods
   FEM method; the code can perform ordinary first order analysis and also second order
   analysis in which it is possible to account for the effects of the nodal displacements in
   the determination of the internal forces. In addition, "Cosmos/M" takes into account of
   material non-linearities and can also perform dynamic analysis.

3 - ULS analysis methods for "sway frames" according to the EC3 (section 5.2.6.2)
   The first order analysis enables to adopt the "Sway mode buckling lengths" method
   (SMBL) and the "Moment amplification" method (MAM).
   The second order analysis enables the "Rigorous second order" method.

4 - Structural model generation - Input data mode
   The model and the input parameters are generated completely interactively. During the
   input process the structure (members, supports, sections, materials, loads, etc) is
   displayed, enabling the designer to check the structural model.

5 - Catalogues of components in the data base
   Not implemented for steel structures.

6 - Automatic member checks
   Not implemented for steel structures.

7 - Outputs
   It is possible to print out the results in tabular form. In addition, the results of the
   calculation can be displayed graphically (deflected structure, internal forces diagrams
   for a member and for the whole structure, support reactions).
1 - **Type of program and structures handled by the code**
   The code is specific for 2D or 3D beam-column frames.

2 - **Analysis methods**
   Stiffness method; the code can perform ordinary first order analysis and also second order analysis (only 2D) in which it is possible to account for the effects of the nodal displacements in the determination of the internal forces. In addition, "Fastsolve" performs (only 2D) elasto-plastic analysis with the "plastic hinge" concept.

3 - **ULS analysis methods for “sway frames” according to the EC3 (section 5.2.6.2)**
   The first order analysis enables to adopt the "Sway mode buckling lengths" method (SMBL) and the "Moment amplification" method (MAM).
   The second order analysis enables the "Rigorous second order" method.

4 - **Structural model generation - Input data mode**
   The input data is given interactively, with the use of input screens and pulldown menus. During the input process the structure (members, supports, sections, materials, loads) is displayed, enabling the designer to check the structural model.

5 - **Catalogues of components in the data base**
   The code has a number of catalogue files for I, H, channel, CHS, RHS sections that give automatically the member properties necessary for the analysis.

6 - **Automatic member checks**
   "Fastsolve" performs the calculation of the cross sectional normal and shear stresses that result from the given load combinations. Neither checking routines nor redesign tools are implemented.

7 - **Outputs**
   It is possible to print out the results in tabular form. In addition, the results of the calculation can be displayed graphically (deflected structure, internal forces diagrams for a member and for the whole structure, support reactions).
**SECOND ORDER COMPUTER PROGRAMS**

<table>
<thead>
<tr>
<th>NAME OF CODE:</th>
<th>COMPANY:</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-DEAS</td>
<td>SDRC - USA</td>
</tr>
</tbody>
</table>

**TYPICAL HARDWARE CONFIGURATION:**
- Workstation

**OPERATING SYSTEM:**
- Unix

<table>
<thead>
<tr>
<th>OPERATIONAL FEATURES:</th>
</tr>
</thead>
</table>

1. **Type of program and structures handled by the code**
   - The code is a multi-purpose for 2D or 3D models; these can be generated from approximately 50 types of different elements. The non-linear analysis are performed by the "I-DEAS Model Solution Nonlinear" while the creation of the structural model is performed by the pre-processor "I-DEAS Finite Element Modelling;"

2. **Analysis methods**
   - FEM method; the code can perform ordinary first order analysis and also second order analysis in which it is possible to account for the effects of the nodal displacements in the determination of the internal forces. In addition, "I-DEAS" performs elasto-plastic, strain hardening, elastic visco-plastic analysis; plasticity is supposed to be diffused along the element (and not concentrated in a single section as in the "plastic hinge" approach).

3. **ULS analysis methods for "sway frames" according to the EC3 (section 5.2.6.2)**
   - The first order analysis enables to adopt the "Sway mode buckling lengths" method (SMBL) and the "Moment amplification" method (MAM).
   - The second order analysis enables the "Rigorous second order" method.

4. **Structural model generation - Input data mode**
   - The model and the input parameters are generated interactively by the "I-DEAS Finite Element Modelling". During the input process the structure (members, supports, sections, materials, loads, etc) is displayed, enabling the designer to check the structural model.

5. **Catalogues of components in the data base**
   - For steel applications, the specific module for steel frames "I-DEAS Beam Modelling" has a number of catalogue files for I, H, channel, CHS, RHS sections that give automatically the member properties necessary for the analysis.

6. **Automatic member checks**
   - "I-DEAS Beam Modelling" module performs the calculation of the cross sectional normal and shear stresses that result from the given load combinations. In addition, the code performs local resistance and buckling checks highlighting the elements that pass these verifications and those that fail at least one. Finally, the program has re-design automatic facilities in which the member section is automatically replaced by a new section that matches the local check requirements.

7. **Outputs**
   - It is possible to print out the results in tabular form. In addition, the results of the calculation can be displayed graphically (deflected structure, internal forces diagrams for a member and for the whole structure, support reactions).
297
1 - Type of program and structures handled by the code
The code is specific for 2D beam-column frames.
The connections between elements may be: perfect restraint, perfect hinge, or semi-rigid connection. Similarly for the supports, these can be fixed, free or semi-rigid. It is important to add that all types of loads have been included; a limitation consists in the fact that "PEP micro" can only deal with a single load combination at a time in each different analysis.

2 - Analysis methods
FEM method; the code can perform ordinary first order analysis and also second order analysis in which it is possible to account for the effects of the nodal displacements in the determination of the internal forces. "PEP Micro" calculates also the "elastic critical load factor" of the structure. In addition, the code performs elasto-plastic analysis in conformity with the "plastic hinge" concept.

3 - ULS analysis methods for "sway frames" according to the EC3 (section 5.2.6.2)
The first order analysis enables to adopt the "Sway mode buckling lengths" method (SMBL) and the "Moment amplification" method (MAM).
The second order analysis enables the "Rigorous second order" method.

4 - Structural model generation - Input data mode
The input data is transferred by an ASCII input file; the data description is performed by means of explicit commands and key words. For nodes and elements there are a number of possibilities for automatic generation. Before running the analysis, the code performs a syntactic processing of the data in order to detect any error in the data. If an error is located, "PEP Micro" indicates its position and the cause.

5 - Catalogues of components in the data base
The code has a number of catalogue files for I and H sections that give automatically the member properties necessary for the analysis.

6 - Automatic member checks
Not yet implemented for steel frames.

7 - Outputs
An "output file" supplies a complete set of results of the analysis (e.g. node displacements, member forces, plastic events, etc.). In addition, the results of the calculation can be displayed graphically (deflected structure, internal forces diagrams for a member and for the whole structure, simulation of the chronology of the plastic hinges, etc.).
1 - *Type of program and structures handled by the code*
   "Sand" is a specific computer program for 2D or 3D beam-column frames

2 - *Analysis methods*
   FEM method; the code can perform ordinary first order analysis and also second order analysis in which it is possible to account for the effects of the nodal displacements in the determination of the internal forces. In addition, "SAND" performs elasto-plastic analysis.

3 - *ULS analysis methods for "sway frames" according to the EC3 (section 5.2.6.2)*
   The first order analysis enables to adopt the "Sway mode buckling lengths" method (SMBL) and the "Moment amplification" method (MAM).
   The second order analysis enables the "Rigorous second order" method.

4 - *Structural model generation - Input data mode*
   The input parameters are given with an input ASCII file. The dead loads are generated automatically and the wind loads semi-automatically.

5 - *Catalogues of components in the data base*
   Not yet implemented for steel frames.

6 - *Automatic member checks*
   Not yet implemented for steel frames.

7 - *Outputs*
   It is possible to print out the results in tabular form. In addition, the results of the calculation can be displayed graphically (deflected structure, undeflected shape, internal forces diagrams for a member and for the whole structure, support reactions).
E.C.S.C. 7210-SA/419
EUROCODE 3 "SWAY FRAMES"

SECOND ORDER COMPUTER PROGRAMS

NAME OF CODE: 3D Smart
COMPANY: Dlubal Eng. SW - Germany

TYPICAL HARDWARE CONFIGURATION: PC compatible 286 or above, 640 Kb RAM
OPERATING SYSTEM: MS-Dos

OPERATIONAL FEATURES:

1 - Type of program and structures handled by the code
The code is specific for 2D or 3D beam-column frames. The connections between elements may be: perfect restraint or perfect hinge. Similarly for the supports, these can be fixed or free. In addition, the code includes spring element releases and elastic foundation elements. It is important to add that all types of loads have been included.

2 - Analysis methods
FEM method; the code can perform ordinary first order analysis and also second order analysis in which it is possible to account for the effects of the nodal displacements in the determination of the internal forces. In addition, "3D Smart" evaluates the overall stability of the structure calculating the elastic critical load factor of the structure.

3 - ULS analysis methods for "sway frames" according to the EC3 (section 5.2.6.2)
The first order analysis enables to adopt the "Sway mode buckling lengths" method (SMBL) and the "Moment amplification" method (MAM). The second order analysis enables the "Rigorous second order" method.

4 - Structural model generation - Input data mode
The input data is given interactively with the use of 15 input screens ("masks") and pulldown menus. There are a number of generation and duplication possibilities that facilitate this task. During the input process the structure (members, supports, etc) can be displayed for verification together with the element properties and the loads. In addition, before running the analysis, the code performs a syntactic processing of the data in order to detect any error in the data. If an error is found, a message is displayed and the cursor indicates the position of the error.

5 - Catalogues of components in the data base
The code has a number of catalogue files for I, H, channel, angle, CHS, RHS sections that give automatically the member properties necessary for the analysis.

6 - Automatic member checks
Not yet implemented for steel frames.

7 - Outputs
It is possible to print out the input data and results in tabular form. In addition, the results of the calculation can be displayed graphically (deflected structure, internal forces diagrams for a member and for the whole structure, support reactions, etc.).
Hereafter is given some information about the Software Companies:

<table>
<thead>
<tr>
<th>Program</th>
<th>Company name</th>
<th>Address</th>
<th>Tel. No.</th>
<th>Fax No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABAQUS</td>
<td>HIBBITT, KARLSSON &amp; SORENSEN, INC.</td>
<td>1080 Main Street, Pawtucket, RI 02860, USA</td>
<td>(1) (401)7274200</td>
<td>(1) (401)7274208</td>
</tr>
<tr>
<td>ADINA</td>
<td>ADINA R&amp;D, INC.</td>
<td>71 Elton Avenue, Watertown, MA 02172, USA</td>
<td>(1) (617)9265199</td>
<td>(1) (617)9260238</td>
</tr>
<tr>
<td>ANSYS</td>
<td>SWANSON ANALYSIS SYSTEMS, INC.</td>
<td>Johnson Road, P.O.Box 65, Houston, PA 15342-0065, USA</td>
<td>(1) (412)7463304</td>
<td>(1) (412)7469494</td>
</tr>
<tr>
<td>COSMOS/M</td>
<td>STRUCTURAL RESEARCH &amp; ANALYSIS CORP.</td>
<td>New Street, Pudsey, Leeds, West Yorkshire LS28 8AQ, ENGLAND</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FASTSOLVE</td>
<td>CSC LTD</td>
<td>2000 Eastman Drive, Milford, OH 45150</td>
<td>(1) (513)5762400</td>
<td></td>
</tr>
<tr>
<td>I-DEAS</td>
<td>SDRC STRUCTURAL DYNAMICS RESEARCH CORP., USA</td>
<td>260 Sheridan Avenue, Suite 309, Palo Alto CA 94306, USA</td>
<td>(1)(415)3296800</td>
<td></td>
</tr>
</tbody>
</table>
Fax No. : (1)(415)3235892

• Program : PEP MICRO
Company name : CTICM
Address : Domaine de Saint-Paul - B.P.n.1, 78470 Saint Rémy-les-Chevreuse - FRANCE
Tel. No. : (33)(1)30852000
Fax No. : (33)(1)30527538

• Program : SAND
Company name : FITZROY COMPUTER SYSTEMS LTD
Address : 50 Fairmile Lane, Cobham, Surrey KT11 2DF, ENGLAND
Tel. No. : (44)(932)65912
Fax No. :

• Program : 3D SMART
Company name : INGENIEUR-SOFTWARE DLUBAL GMBH
Address : Am Zellweg 2, D-93464 Tiefenbach, GERMANY
Tel. No. : (49)(967)31775
Fax No. : (49)(967)31770
CORDIS represents a central source of information crucial for any organisation - be it industry, small and medium-sized enterprises, research organisations or universities - wishing to participate in the exploitation of research results, participate in EU funded science and technology programmes and/or seek partnerships.

CORDIS makes information available to the public through a collection of databases. The databases cover research programmes and projects from their preparatory stages through to their execution and final publication of results. A daily news service provides up-to-date information on EU research activities including calls for proposals, events, publications and tenders as well as progress and results of research and development programmes. A partner search facility allows users to register their own details on the database as well as search for potential partners. Other databases cover Commission documents, contact information and relevant publications as well as acronyms and abbreviations.

By becoming a user of CORDIS you have the possibility to:

- Identify opportunities to manufacture and market new products
- Identify partnerships for research and development
- Identify major players in research projects
- Review research completed and in progress in areas of your interest

The databases - nine in total - are accessible on-line free of charge. As a user-friendly aid for on-line searching, Watch-CORDIS, a Windows-based interface, is available on request. The databases are also available on a CD-ROM. The current databases are:

News (English, German and French version) - Results - Partners - Projects - Programmes - Publications - Acronyms - Comdocuments - Contacts

CORDIS on World Wide Web

The CORDIS service was extended in September 1994 to include the CORDIS World Wide Web (WWW) server on Internet. This service provides information on CORDIS and the CORDIS databases, various software products, which can be downloaded (including the above mentioned Watch-CORDIS) and the possibility of downloading full text documents including the work programmes and information packages for all the research programmes in the Fourth Framework and calls for proposals.

The CORDIS WWW service can be accessed on the Internet using browser software (e.g. Netscape) and the address is: http://www.cordis.lu/

The CORDIS News database can be accessed through the WWW.

Contact details for further Information

If you would like further information on the CORDIS services, publications and products, please contact the CORDIS Help Desk:

CORDIS Customer Service
B.P. 2373
L-1023 Luxembourg

Telephone: +352-401162-240
Fax: +352-401162-248
E-mail: helpdesk@cordis.lu
WWW: http://www.cordis.lu/
The present work deals with the analysis and verification, according to the Eurocodes, of a particular class of steel structures known as 'sway frames' in which the nodal horizontal displacements produce non-negligible additional internal moments (P-Δ effects) that have to be summed up to those deriving from a first-order analysis.

The final result of this project is the draft of a practical design handbook for structural engineers dealing with the design of steel sway frame structures according to Eurocode 3 (EC3). The contents of this document are the design calculation rules for this category of structure. In addition, some indications are provided as to the use of popular general-purpose computer codes available in the European market for a reliable and efficient design of sway frame systems. Finally, a number of calculation examples of 2D frames have been included.

From a practical point of view, this research has shown that common types of steel structures such as industrial buildings, multi-storey car parks, warehouse buildings and small civil buildings subject to usual vertical loads may fail to fulfil the 'non-sway' criterion and thus have to be defined as sway frames.

The results have shown that for asymmetrical plane frames the rules of EC3 for the simplified 'sway/non-sway' classification and for the indirect second-order elastic analysis methods have to be reconsidered in order to make them clearer and to improve their level of accuracy. Further studies are needed to develop the background to simple yet reliable design criteria.