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Contents

SEISMIC DESIGN IN PLANT CONSTRUCTION
Shortcomings of Eurocode 8

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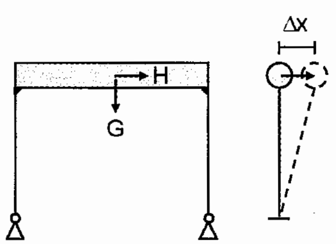
INTRODUCTION

In designing light steel structures against seismic actions very often ductility class 'low' is used in structural analysis. This gives maximum base shear but minimum effort for the structural engineer and the builder. In many cases this procedure provides economic structures because only a few structural members such as braces and adjacent columns have to be upsized. However, in plant construction type and position of the braces often need to be irregular due to operational reasons, so that the recommendations in Eurocode 8 (EC8) [1] concerning simplicity and uniformity can hardly be met and parallel walls might have different stiffnesses and strengths. In the following it is shown how to design high ductility plant structures, which are capable of withstanding seismic action although some rules of EC 8 [1] are violated.

1 GENERAL

1.1 Dynamic modelling

Discrete dynamic models are used in seismic design. Uniformly distributed mass (e.g. from slab) is hereby concentrated at specific locations of the structure. Each concentrated mass appears then as a rigid body with 3 degrees of freedom or 6, if Cosserat theory is used. So the number of degrees of freedom with n masses is 6 times n.
Analytical models are further simplified to comprehend the dynamic behaviour of complex structures. For multi-story buildings the mass is concentrated in the particular floor levels. In addition, plane structures are assumed for symmetric and uniform buildings where vertical and torsional vibrations are neglected. So only the horizontal displacements have to be considered and the number of degrees of freedom is equal to the number of concentrated masses.
So a plane single-storey frame, as shown in Fig. 1 (a), has only one degree of freedom.



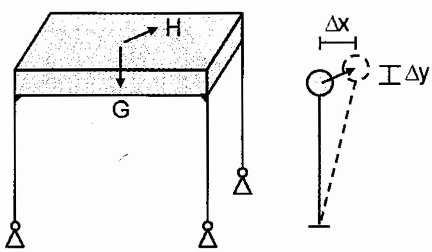


Fig. 1. Dynamic modelling of a single-storey frame

1.2 Three-dimensional structures

Since buildings are three-dimensional structures, anyhow, the second horizontal direction has also to be considered, as shown in Fig.1 (b). Usually seismic design can be carried out separately for both main directions perpendicular to each other. So the problem is reduced to two plane frame calculations, one in x- and one in y-direction. An independent treatment of actions on the structure along both main axes implies that no torsion occurs. Generally, this is the case for compact buildings with uniform distribution in vertical and symmetrical distribution of masses and bracings in horizontal projection.

1.3 Non-uniform structures

Asymmetric mass distribution and non-regular structures are daily occurrence in plant construction. So an offset between the centre of mass and the centre of stiffness (shear centre) cannot be avoided in most cases. During an earthquake, this eccentricity causes vertical torque, which has to be considered in seismic design.

2 SEISMIC DESIGN ACCORDING EUROCODE 8

2.1 Torsional effects

To cover uncertainties in the location of masses and in the spatial variation of the seismic motion, the calculated centre of mass at each floor *i* shall be considered displaced from its nominal location in each direction by an accidental eccentricity of 5% , see section 4.3.2 eq. (4.3) of EC 8 [1].
Beyond that, EC 8 [1] section 4.3.3.1 (8) gives different conditions concerning the uniformity and maximum allowed eccentricity. Hereby, a linear-elastic analysis with two plane models in each main direction is possible. But if the eccentricity of masses exceeds the given limits the seismic loads have to be multiplied by 1.25. In addition, torsional effects are accounted for by an enlargement factor δ for the loads, see section 4.3.3.2.4 eq. (4.12) of EC 8 [1]. If the analysis is performed using two planar models, one for each main horizontal direction, double eccentricity and an increased factor δ has to be applied. Assuming the considered element is a bracing at the outer side of the building ($x = L_e / 2$) and an analysis by two separated planar models is performed, internal loads have to be increased by a factor $1.25 \cdot 1.6 = 2$ together with a 10 % eccentricity of the storey mass from its nominal location.
On the one hand, superposition of loads is only necessary for bracing systems with a shared corner column. For systems with multiple fields and intermediate supports, as shown in Fig. 2, the loads out of different directions and the caused torsion do never affect the same column at the same time. Only if braces of both main directions affect the same column, superposition of loads becomes decisive. Since this is hardly ever the case in steel construction, the overall increase of the internal forces as recommended by EC 8 [1] leads to oversized structures. Torque modes are then loading bracings in parallel outer walls with opposite sign and the base shear to be taken from the bracing can not be bigger than from a translatory sway-mode in the same direction.

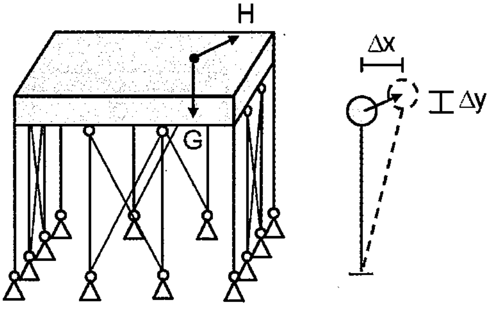


Fig. 2. Single-storey frame with multiple columns and fields

On the other hand, steel structures for industrial plants are 3-D-modelled for structural design, anyway. If natural frequencies are determined, modes are obtained for predominant sway in two main directions and predominant twist around a vertical axis.

2.2 Ductility classes

In ductility class 'low' (DCL) only the natural ductility with low dissipative capacity is taken into account. The structure behaves predominantly elastic and members only show limited non-linear behaviour [2]. Generally, a behaviour factor q = 1.5 may be applied. Since low q values enlarge the design spectrum S_d and therefore the seismic loads acting on the structure, a design by means of ductility class 'low' is always an approximation at the cost of material optimization. Because of the supposed lower calculation effort, structural engineers prefer this way, anyhow [3].

In ductility classes 'medium' (DCM) and 'high' (DCH) special design features make use of distinctive dissipative behaviour due to plastic action of the members. Sufficient plastic deformation capacity is provided by dissipative members and by capacity design of joint members and connections. For ductility class 'medium' (DCM) behaviour factors up to q = 4 may be applied, for ductility class 'high' (DCH) even up to q = 8 (= 5 * 1.6) [4]. In comparison to the behaviour factors possibly applied for ductility class 'low' (DCL) this reduces the calculative seismic loads approximately by a factor of 2.5 to 5. The determination of a plastic mechanism as necessary for dissipative design with sufficient overstrength of members which do not take part in the plastic mechanism is complex. For plant constructions with non-uniform layout and not clearly separated floors, often the possible material savings do not compensate time and effort for design.

Generally, for non-regular structures the above behaviour factors have to be multiplied with 0.8 (EC8 4.2.3.1 (7) [1]); interesting that in the German text this applies only for structures, which are non-regular in elevation.

2.3 Energy dissipation capacity

X-bracings with only tension struts are not sensible for seismic design, since during change of excitation direction, the discontinuous load distribution utilizes and endangers the stability of the profiles. This effect might be countered by a sophisticated design of constructional details, e.g. a bolted connection in the middle of the bracing. Generally, eccentric connections of the braces to the beams have figured out to be more advantageous concerning the plastic behaviour [5]. For 'high' ductility design (DCH) this is also reflected in the more favourable behaviour factor of q = 6 for frames with eccentric bracings instead of q = 4 for concentric diagonal bracings. In ductility class 'medium' (DCM) for both cases behaviour factor q = 4 has to be applied, see EC 8 section 6.3.2 and Tab.6.2 [1]. More detailed information can be found in [6].

In rigid frames the dissipative zones or plastic hinges respectively must lay in beams or in the beam-to-column connection, see Fig. 3. Hinges in columns are also possible but only at the column base or at the top of single-storey buildings, since otherwise the plastic deformation leads to a kinematic unstable system. Further remarks on ordinary building constructions are given in [7] and [8].

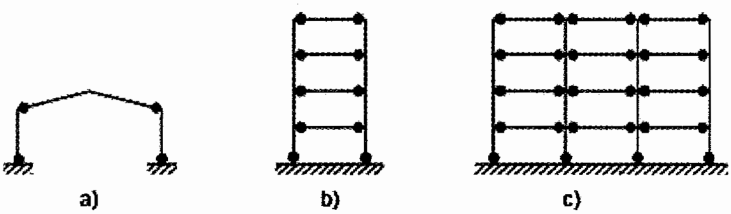


Fig. 3. Rigid frames with dissipative zones in beams and column base joints; taken from Figure 6.1 of EC 8 [1]

The challenge in design is to find the balance between stiffness and load bearing capacity, so that the plastification actually occurs in the beams. This can be realized by large profile dimensions for the columns. But nevertheless to ensure sufficient overstrength especially for non-uniform structures or buildings, each member and connection has to be single checked, which implies an enormous effort to the structural engineer.

2.4 Jump of behaviour factors

In plant structures the use of only one type of bracing system does not fulfil the special demands, so that bracing systems are often mixed up. Taking the example of the forming station in Fig. 5, in the second storey 16 fields exist in longitudinal direction and only one of them is braced with concentric diagonals. The others are moment resisting frames with dissipative zones in beams. In ductility class 'high' (DCH) for concentric diagonal bracings a behaviour factor q = 4 has to be applied, whereas for rigid frames a behaviour factor q = 6 (= 5 • 1.2) is valid. The common engineering procedure would now imply to calculate on the safe side and therefore to take behaviour factor q = 4. The large jumps of the value of the behaviour factors (here from 4 to 6) as well as the "black or white" classification leaves little play left for the designer. A more sensitive approach might be a linear interpolation between the corresponding behaviour factors depending on the percentage of the bracing system of the whole structure in one main direction. That would mean, that even if half of the fields have concentric diagonal bracings, still a behaviour factor of q = 5 might be applied, see Fig.4.

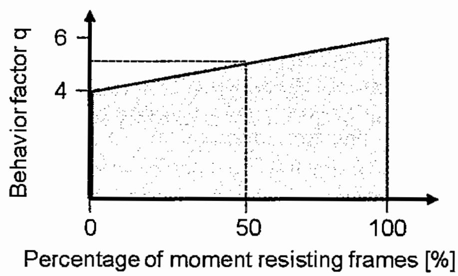


Fig. 4. Example for a possible linear approach of the behaviour factor q in mixed systems

3 EXAMPLE – FORMING STATION

3.1 Subject matter

The following example shows a forming station for the production of OSB-boards, representing a typical plant structure, which was planned and erected by Dieffenbacher GmbH once in Bolderaja, Latvia, and once in Brasov, Rumania.

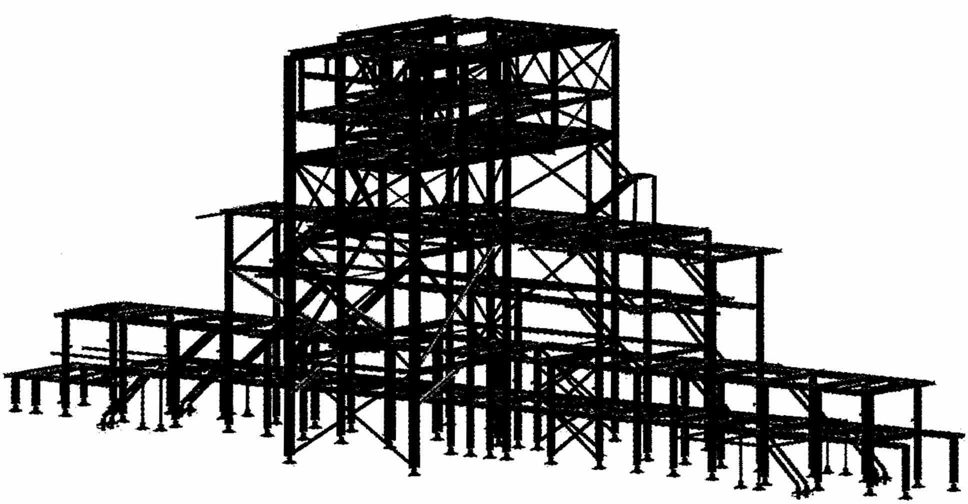


Fig. 5. Structural model of a forming station for OSB-board production

The structural framing model of the forming station is shown in Fig. 5, where the irregular structure in vertical and horizontal projection comes out clearly. While in Bolderaja no seismic design had to be carried out, in Brasov accelerations of 2 m/s^2 had to be considered.

3.2 Preliminary study

To support the decision of the constructor, a preliminary study concerning the design method was carried out. For this purpose a 3-storeyed frame as a part of the forming station subjected to real loads was analysed for different design methods [9]. The comparison of the steel masses needed depending on the used ductility class behaviour factor q is given in *Table 1.* Although the overall design was carried out according to EC 8 [1], some essential requirements of the Romanian standard P100 [10] were considered, a code that is somewhat harmonized to EC8 [1].
As a basis the structural steel masses without consideration of earthquake was calculated to 2.6 tons. A design using ductility class 'low', which means a behaviour factor q = 1.0 according to P100 [10], resulted in a structural steel mass of 10.0 tons, which means an increase of 400 %, see *Table 1.* By using X- bracings and ductility class 'medium', which applies to a behaviour factor q = 4.0, this mass can be reduced by the half. A design with K-bracings, which is not allowed according to EC8 [1], and behaviour factor q = 2.0 led to an intermediate value of 7.2 tons.

Table 1. Comparison of structural steel masses of a 3-storeyed frame depending on design method

Ductility class	Behaviour factor q	Steel mass [tons]
No seismic loads	-	2.6
Romanian DCL	1.0	10.0
Romanian DCM (K-bracing)	2.0	7.2
Romanian DCM (X-bracing)	4.0	5.0

As already mentioned this study was based on a simplified model where the percentage of the bracing system was relatively high. For a whole plant structure the steel mass increase due to seismic loads is not expected to be that high. Nevertheless, the results of this study give an approximation about the influence of the behaviour factor q on the structural steel mass.

3.3 Final design

The forming station in Bolderaja was designed and erected at first. Without seismic design the total mass of the steel structure including grating and guard railing was approximately 150 tons. For the Brasov forming station, the structure erected in Bolderaja was taken as a basis and adapted for seismic design.
In a first step, seismic assessment was carried out for ductility class 'low' and behaviour factor q = 1.0 in order to minimize engineering effort. The resulting steel mass for this solution was about 380 tons. The enormous increase of the steel mass can be explained by the fact that not only bracing elements, but also connections and all members, which participate in load distribution, have to be upsized. Usual building constructions have to transfer vertical loads primarily. Under seismic impact also beams, which were initially designed for bending, have to bear axial forces and participate in the horizontal load transfer.
The final design of the forming station in Brasov was then carried out using energy dissipation capacity, fulfilling the requirements of the Romanian standard P100 [10]. In doing so the structural steel mass could be reduced by 100 tons to approximately 280 tons, anyhow.

4 SUMMARY

When dealing with big masses at high elevations, which is a common task in plant construction, it can be more efficient to use ductility class 'high' in structural analysis. This gives minimum base shear and therefore minimum structural weight but requires maximum effort in analysis, design, production, quality management and supervision. The final decision lies in the building owner's interest and the pros and cons between optimum structural weight and minimum engineering effort has to be weight with care.

The example of the forming station has shown that with a high ductility design in plant construction the structural steel mass can be reduced to about a third to a fourth. Depending on size of the structure and total construction weight, as well as the expert knowledge of the structural engineer, a seismic design according to ductility class 'medium' or 'high' can be economic.

EC8 [1] does not offer an implementation concept for the handling of structures with mixed bracing systems. Due to the increments of the behaviour factor's values, mixed bracing systems as commonly used in plant structures are often poorly classified. A linear interpolation of the corresponding behaviour factors depending on the percentage of the type of bracing systems used can provide a more efficient design.

It has been shown that an analysis by separated planar models for the two main directions leads to intolerable oversized structures when following the rules of EC 8 [1]. The required increase of seismic loads due to non-regularity is partly not comprehensible and wrong in the case of systems with multiple fields and intermediate supports, as shown in Fig. 2. For non-uniform structures 3-D-modelling is the only way to proceed in seismic design.

It should be also mentioned here, that typical designations for dynamic parameters, as long time used in textbooks, are not adopted or changed in EC8 [1]. For engineers familiar with dynamics and its nomenclature, the handling of EC8 [1] is confusing.

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