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Design guidelines for precast structures with cladding panels

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Foreword

This document has been drafted within Work-Package WP6, “Derivation of design guidelines” of the SAFECLADDING Project (FP7-SME-2012-2 Programme, Research for SME associations - Grant agreement n. 314122).

The SAFECLADDING project (Improved Fastening Systems of Cladding Wall Panels of Precast Buildings in Seismic Zones) is a comprehensive research and development action performed by a group of European associations of precast element producers and industrial partners with the assistance of a group of RTD providers.

The partners were: BIBM, European Federation for the Precast Concrete industry, represented by Dr. Alessio Rimoldi; ASSOBETON, National Italian Association of Precast Concrete Producers, represented by Dr. Antonella Colombo; TPCA, Turkish Precast Concrete Association, represented by Dr. Bulent Tokman; ECS, European Engineered Construction System Association, represented by Dr. Thomas Sippel; POLIMI, Politecnico di Milano, represented by Prof. Fabio Biondini; UL, University of Ljubljana, represented by Prof. Matej Fischinger; NTUA, National Technical University of Athens, represented by Prof. Ioannis Psycharis;, ITU, Istanbul Technical University, represented by Prof. Faruk Karadogan; JRC, Joint Research Centre - Elsa Laboratory, represented by Dr. Paolo Negro; B.S. Italia, represented by Mr. Sergio Zambelli; YAPI, Yapi Merkezi Construction and Industry Inc, represented by Mr. Orhan Manzac; ANDECE, Asociación Nacional de la Industria del Prefabricado de Hormigón, represented by Mr. Alejandro Lopez Vidal.

Dr. Alessio Rimoldi served as the coordinator of the SAFECLADDING project. And Prof. Giandomenico Toniolo was charged with the technical management of the SAFECLADDING project and was the Work-Package leader for the Work-Package WP6 “Derivation of design guidelines”, of which this document represents the final outcome.

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Abstract

The current design practice of precast buildings is based on a frame mode, where the peripheral cladding panelsenter only as masses without anystiffness. The panels are then connected to the structure with fastenings dimensioned with a local calculation on the basis of their mass for anchorage forces orthogonal to the plane of the panels.

This design approach does not work, as it was recently dramatically shown by several recent violent shakes, like L'Aquila (Italy) in 2009, Grenada (Spain) in 2010, and Emilia (Italy) in 2012. The panels, fixed in this way to the structure, come to be integral part of the resisting system, conditioning its seismic response. The high stiffness of this resisting system leads to forces much higher than those calculated from the frame model. These forces are related to the global mass of the floors and are primarily directed in the plane of the walls.

Furthermore, the seismic force reduction in the type of precast structures of concern relies on energy dissipation in plastic hinges formed in the columns. Very large drifts of the columns are needed to activate this energy dissipation foreseen in design. However, typically, the capacity of the connections between cladding and structure is exhausted well before such large drifts can develop. Therefore, the design of these connections cannot rely on the seismic reduction factor used for design of the bare structure.

New technological solutions for connectors with proper design approaches were urgently required. The research project SAFECLADDING was thus aimed at investigating, by means of a balanced combination of experimental and analytical activity, the seismic behaviour of precast structures with cladding wall panels and at developing innovative connection devices and novel design approaches for a correct conception and dimensioning of the fastening system to guarantee good seismic performance of the structure throughout its service life.

The final outcome of the SAFECLADDING project is represented by a set of documents providing the design guidelines produced by the consortium. The guidelines have a theoretical derivation supported by the experimental results of the testing campaigns and numerical simulations performed within the project. General know-how on production practice and international literature on the subject have been also considered.

The present document provides the design guidelines for precast structures with cladding panels. A companion document provides the design guidelines for the wall panel connections.

Introduction

The structural analysis of precast buildings under seismic action shall properly take into account the role of cladding panels in the seismic response of the overall construction assembly. To this end the models used in the analysis shall represent as close as possible the real arrangement of the construction, including panels and relative connections. Specific indications are given in the following chapters with reference to the four systems of cladding connections quoted in 0.1 of *DGA*.

Scope

The present document refers to the methods of structural analysis to be applied for the seismic design of precast structures. They can be used for single-storey buildings and, with proper modifications, for multi-storey buildings. Special attention is devoted to the proper representation of cladding panels within the model of the resisting system. Specific indications are given about the level of refinement required for the models used in the analysis of the different solutions.

The precast structures considered are frame systems made of columns and beams connected with horizontal floor diaphragms. In particular the roofs can provide rigid, deformable or null diaphragms. To this frame system, for the peripheral cladding, a set of wall panels is added that, depending on the type of connections to the structure, may not interfere with the frame behaviour or may interfere leading to the interaction between the panels and the frame and to an increased stiffness of the system. In this dual wall-frame system a set of dissipative connections may be present able to attenuate the seismic response. All these structural systems have to be possibly analysed with proper specific calculation models. A wide parametric analysis is shown in Annexes O, A, B. C and D.

Possible internal partitions of the building, made with the same types of wall panels of the peripheral claddings, can be treated in the structural analysis in the same way.

Methods for structural analysis

As primary type of analysis for the current design practice the linear elastic analysis with response spectrum is proposed as regulated by Clause 4.3.3 of *EC8*, where the effects of energy dissipation at the ultimate limit state are represented by the behaviour factor q_p (see 5.11.1.4 of *EC8*). Alternatively, the other types of analysis included in *EC8* can be used according to the requirements of its Clause 4.2.3.

It is taken as ordinary approach that the structural analysis is elaborated by means of common commercial programmes for electronic computation and that the dynamic modal analysis is applied to a spatial model of the structural assembly. Simplified approaches, such as static analysis (lateral force method) and separate plane analyses in the two main directions, can be adopted provided the conditions of regularity summarised in Table 4.1 of *EC8* are fulfilled.

In particular from the analysis the forces and displacements on the connections of the panels are expected for the necessary verifications.

Parameters of seismic behaviour

The following suggestions refer to the modern production of precast concrete structural elements through processes under quality control as regulated by the present European harmonised standards and to the related new constructions.

With reference to the provisions of Chapter 5 of *EC8*, these structures can have all the characteristics to be considered in ductility class high *DCH* (ductile reinforcement, proper detailing, over-proportioned connections). However the use for frame structures of the high q_o factor given by Table 5.1 of *EC8* to this class could lead to an excessive deformability incompatible with the requirements of the damage limitation state, with floor drifts larger than 1%. To fulfil this limit state a larger proportioning in size of the columns could be necessary (corresponding to a lower q_o factor), aware that in this way the isostatic solutions will result over-dimensioned in strength. For all connection systems, a ratio α_u/α_1=1,0 shall be taken.

In any case it is highly recommended to apply the rules of DCH for reinforcement ductility, member detailing and connection over-proportioning (with k_p=1,0). In particular, for a full exploitation of the ductility resources in compression of the longitudinal bars of columns, in the critical regions of the columns a spacing of stirrups $s\leq 3,5\phi$ should be adopted, with ϕ diameter of the longitudinal bars.

Bibliography

Some references are here listed together with the corresponding abbreviated symbols used in the text.

DG0 Design guidelines for connections of precast structures under seismic action (SAFECAST Project), 2014 JRC Scientific and policy reports

DGA Design guidelines for wall panel connections (SAFECLADDING Project), 2016 JRC Technical Reports

EC2 EN 1992-1-1 Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for Buildings, 2004 CEN

PT4 EN 1992-1-4 Eurocode 2: Design of concrete structures – Part 1-4: Design of fastenings for use in concrete, Draft 2014 CEN

EC8 EN 1998-1 Design of structures for earthquake resistance – Part 1: General rules, seismic action and rules for buildings, 2004 CEN

PT3 EN 1998-3 Design of structures for earthquake resistance – Part 3: Assessment and retrofitting of buildings

1. EXISTING BUILDINGS

A structural analysis may be required for the verification of the seismic capacities of existing buildings, in terms of resistance and stability under the expected seismic action. Following the results of this verification, proper interventions of upgrading or retrofitting could be decided. These interventions should be performed according to the provisions of *PT3* and relevant National Annex. When it is technically and economically possible, a retrofitting should be made with the full fulfilment of the code requirements for new constructions. However, when *PT3* and the National Annex permit, the upgrading may correspond to an improvement of the capacity that doesn't reach the level required in the zone for the new constructions. In such a case lower return periods of the earthquake can be used leading to lower peak ground accelerations compared to those used for the new buildings in the zone. At the present some countries allow the reduction factors as low as 0,6. In all cases the damage limitation requirement can be disregarded.

1.1 General indications on seismic design

In the case of existing buildings originally not designed for seismic action, the analysis should be based on the assumption of a low ductility class for which a behaviour factor $q=1,5$ shall be adopted. If the existing buildings were designed for seismic action, higher behaviour factors can be used, if justified by the analysis and structural details used at the time of construction.

Adequate seismic design procedure is proposed in Section 2.3 (see flow-chart in Figure 2.6). It is assumed, however, that the procedure will frequently demonstrate the inadequacy of the panel connection systems in existing buildings. In such a case proper upgrading or retrofitting interventions should be made.

1.2 Suggestions for the structural model

Any type of structural model and analysis foreseen in *PT3* can be used for the evaluation of the existing precast buildings. While the equivalent elastic modal spectrum analysis is the first choice in the current design practice, more refined inelastic analysis might be warranted in some cases due to the highly complex non-linear behaviour considering panel-to-structure interaction (see Section 1.1 and Chapter 2).

In the general case of existing precast structures, the frame or dual wall-frame analysis will be performed by means of electronic computation following the basic indications stated in 0.2. The ordinary methods for the formulation of the calculation model will be applied with the specific pertinent indications given in 2.2 for current systems, 3.2 for isostatic systems or in 4.2 for integrated systems of connections.

1.3 Conditions for strengthening interventions

Any intervention of upgrading or retrofitting of panel connections on existing buildings should be made only when the adequacy of all the remaining parts of the structure has been verified to be compliant with the requirements of the chosen level of seismic resistance (see paragraph 1).

Following the experience of recent earthquakes, in addition to the failure of inadequate panel connections, another widespread fatal deficiency has been noticed in buildings not designed for seismic action: the loss of bearing of beams and floor elements in dry simple supports that entrust the transmission of horizontal forces only to friction without mechanical restrainers. Proper connection devices are needed able to transfer horizontal forces also in absence of the gravity action. And this is valid also with reference to the possible lateral overturning of beams. These connection devices should be over-dimensioned taking into account the possible stiffening effects of the cladding panels with respect to the response of the bare frame structure.

2. CURRENT FASTENING SYSTEMS

The term "current systems" is used in these guidelines for the precast buildings with the existing fastening systems of cladding panels (see *DGA* for more detailed description), which have been extensively used in the past and are still used at present. The existing buildings of this type are addressed in Chapter 1. In the following the conditions to apply the current fastening systems also to new constructions are specified.

As described in Chapter 1 of *DGA*, the existing design practice for the current systems has been usually based on the model which is not explicitly considering the interaction between the main structural system and the claddings in the plane of the façade. As recent earthquakes and research demonstrated, such approach cannot identify eventual complex interaction between the structure and the panels leading to possible failure of the fastening system and the fall of the panels during strong earthquakes. Nevertheless, some of such systems, in case of small seismic demand and/or structures with large over-strength and stiffness, can provide sufficiently safe design solutions.

Therefore suitable design methodology, which is able to identify when the current systems can be safely used, is provided in these guidelines in the following sections. It can be used for the design of the new structures and also, in the modified version, for the evaluation of the existing ones (see Section 2.3 and Chapter 1).

2.1 General design methodology

These guidelines are strictly limited to those systems, which were investigated well enough to reliably provide safe recommendations for their application and design. The given recommendations are therefore directly applicable only for the current fastening systems described in the associated document *DGA*. When the applied fastening system is different from those presented in *DGA*, the system shall be experimentally and analytically investigated (taking into account the 3D behaviour of the structure) to provide the basic data needed in the proposed methodology. These data include, but are not limited to, the mechanism of the structural-to-panel interaction, deformation and strength capacity, equivalent stiffness, and, in the case when refined inelastic response analysis is chosen, the hysteretic models for the structure and the fastening system.

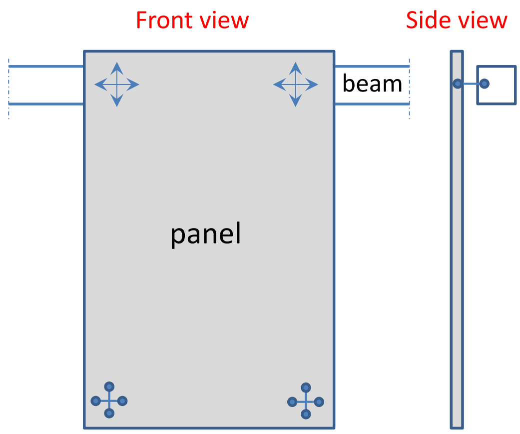


Figure 2.1

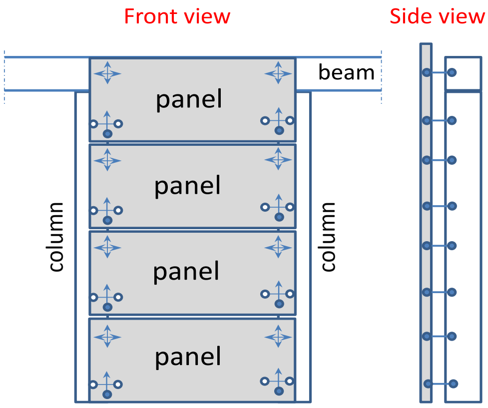


Figure 2.2

Furthermore these guidelines are limited to fastening schemes presented in Figure 2.1 for vertical panels and Figure 2.2 for horizontal panels. In particular the vertical panels are attached to the upper beam with two connections giving bilateral restrain in y (orthogonal) direction and bilateral essentially sliding freedoms in x (horizontal) and z (vertical) directions, while at the base they are supported with two pinned connections providing restrains in all the three directions. Any horizontal panel is attached to the lateral columns with two connections at the upper part similar to the ones of the vertical panels and with two connections at the lower part giving bilateral restrain in y (orthogonal) direction, unilateral support in z (vertical) direction and bilateral partially sliding (friction or "strap" type) freedom in x (horizontal) direction.

The suggested approach has two possible levels of complexity and it is based on the following main considerations:

a) Weak interaction between the panel and the bare frame (i.e. the stiffness of the fastening devices is small compared to the stiffness of the structure itself) may be expected in current systems until certain deformation threshold is exhausted. Until this deformation limit is reached, the system behaves essentially as isostatic and relatively simple traditional structural models can be used, basically neglecting the structure-to-cladding interaction. The relevant deformation capacity of the addressed current systems is provided in *DGA*.

b) After the deformation limit is reached, more complex model should be used considering the interaction between the panels and the bare structure through the fastening system. Relevant input parameters for the addressed current systems are provided in *DGA*.

c) If the more refined model does not prove the adequacy of the system, a different cladding connection system should be chosen for new buildings or a proper upgrading or retrofitting intervention should be made for existing buildings.

Verification of the connections in the direction perpendicular to the plane shall always be done.

The most critical problem in the case of the current systems is their quite limited deformation capacity. Below this limit the current connections behave essentially as isostatic connections.

Since the deformation threshold and the interacting mechanism are highly uncertain, the use of the back-up devices (restrainers – see Clause 1.2 of *DGA*) is always strongly recommended both, for existing and new buildings. A more detailed flow-chart for this methodology is presented in Figure 2.2:

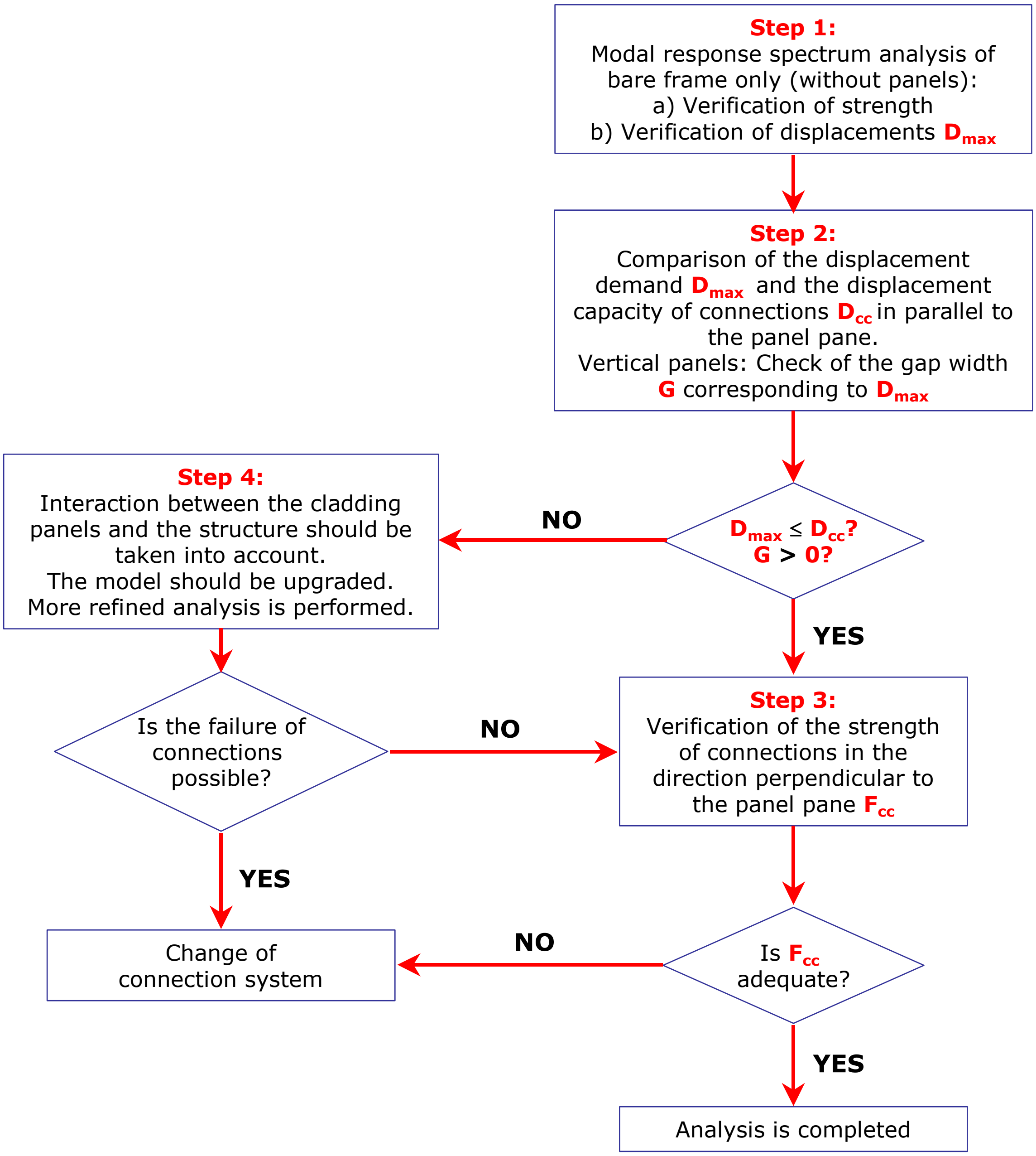


Figure 2.3

2.2 Application procedure

The practical design application of the methodology described in 2.1 can be made through the steps described in the following both for new and existing buildings.

2.2.1 New buildings

Step1:

First the bare frame (without panels) is analysed taking into account the seismic intensity, defined for the particular site according to the requirements of *EC8* and National Annexes. The modal response spectrum analysis is used for the analysis.

It is recommended that the estimation of the required strength of the structure is based on the un-cracked cross-section of columns. When displacements are examined the cracked cross-sections shall be considered.

Note: The resulting increased action accounts for stiffening of the structure due to the interaction with the cladding panels (although this is rather small until the deformation threshold of the connections is not exhausted) and reduces the deformability of the structure.

Step2:

The displacement capacity of the connections in the plane of the panel is compared with the displacement demand defined in Step 1 as it is stated below:

a) Vertical panels: the displacement capacity of the top connections are compared with the displacements of the beam (see Figure 2.4). The size of the gap, corresponding to the displacement demand should be checked according to the procedure presented in clause 2.2.3 of DGA.

b) Horizontal panels: the displacement capacity of the connections is compared with displacements of the columns at the level of these connections (see Figure 2.5).

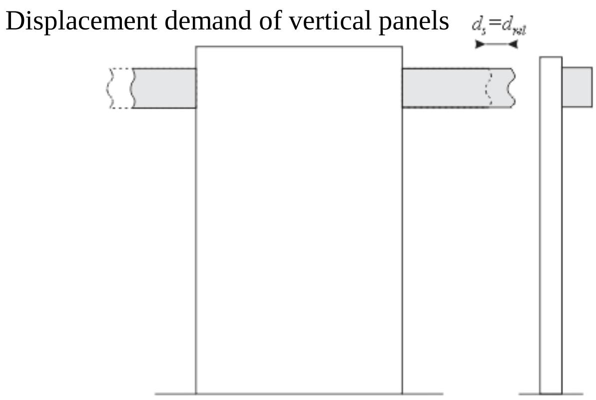


Figure 2.4

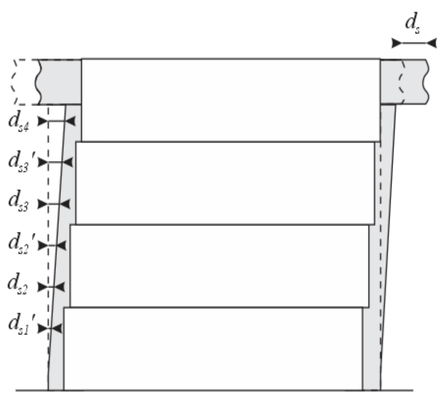


Figure 2.5

If the displacement capacity of vertical panel connections is larger than the displacement demand, the analysis is completed unless the gap between the beam and the panel is closed. If the displacement capacity of horizontal panel connections is larger than the displacement demand, the analysis of this step is completed. In such cases it can be assumed that the interaction between panels and the bare frame is weak.

If the displacement demand exceeds the capacity of the connections and/or the gap between the vertical panels and the beam is closed, the cladding-to-structure interaction shall be considered in the analysis (see Step 4) or different cladding system should be chosen.

Step 3:

In the direction perpendicular to the panel, the strength of all connections shall be verified with respect to the demand, evaluated according to Clause 4.3.5.2 of *EC8*. The capacity of the connections in the direction perpendicular to the panel is estimated according to the data provided by the manufacturer. If the strength of the connections is inadequate, a different connection system should be chosen.

Step4:

The cladding-to-structure interaction is taken into account by means of a more refined structural analysis. Recommendations regarding this analysis are provided in the next Section 2.3.

If the displacement demand exceeds the capacity of the connection and/or the gap between the vertical panel and the beam is closed, a different connection system should be chosen. If not, go to Step 3.

2.2.2 Existing buildings

The proposed analysis procedure can be in principle used for both, new and existing structures. When it is used for existing buildings (see more detailed discussion and recommendations for existing buildings in Chapter 1) the following specifics should be considered:

- The analysis and verification should follow the requirements of PT3.
- In Step 1 the adequacy of the main structural system (bare frame) should be checked first before proceeding to Step 2. The main structural system itself may need upgrading first.

The whole procedure is illustrated in Figure 2.6.

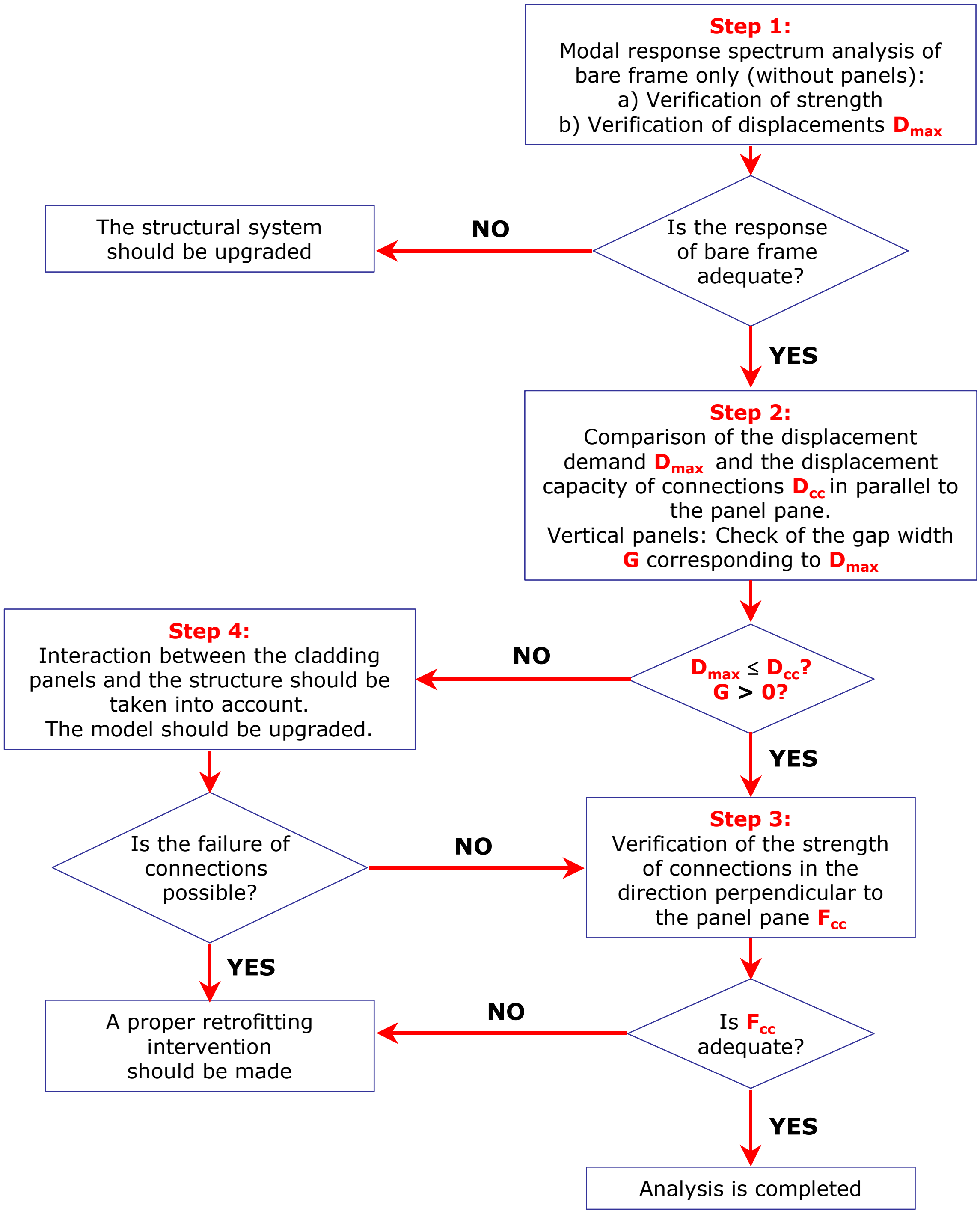


Figure 2.6

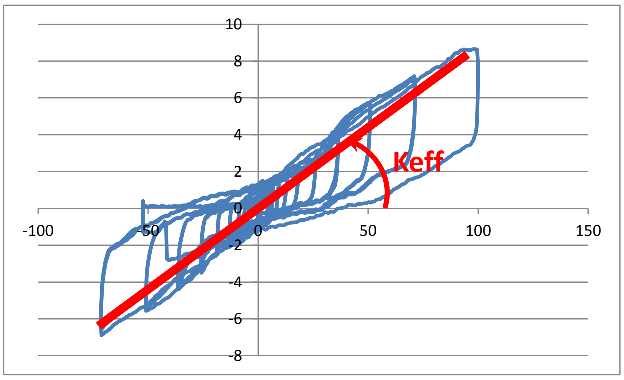
2.3 Refined analysis model

Numerical models of the connections shall be able to describe all important features of their seismic response. For current fastening systems the appropriate hysteretic models presented in Chapter 2 of *DGA* can be used. Such models may imply the nonlinear dynamic analysis, which should be performed according to *EC8*. For regular structures the analysis can be simplified using the nonlinear pushover-based analysis according to the requirements of *EC8*.

The quoted numerical models of connections can be further simplified. An equivalent linear model (see Figure 2.7), considering an increased effective damping ξ_{eff} (hysteretic damping is taken into account), can be used. The damping can be estimated as:

$$\xi_{eff} = \frac{1}{2\pi}\left[\frac{\Sigma E_{D,i}}{K_{eff}\, d_{cd}^2}\right]$$

The variables used in this equation are illustrated in Figure 2.8.



Keff – effective stiffness

Figure 2.7

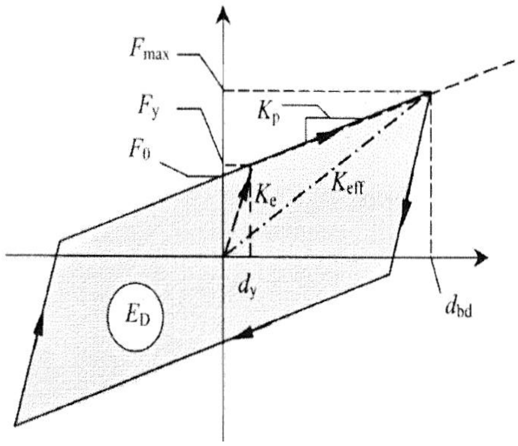


Figure 2.8

3. ISOSTATIC SYSTEMS

For new buildings with isostatic arrangements of wall panel connections, the structural analysis under seismic action can refer to the frame system following the current design practice of such structures. In expectation of large displacements the second order " P/Δ " effects should be taken into account. In addition to the ordinary out-put data used for the verification of member resistance at ultimate limit state, the sliding or rotation displacements shall be provided for the verification of the pertinent capacities of panel connection devices.

3.1 General indications on seismic design

For the one-storey frame systems considered in this chapter, the application of capacity design criteria for connection proportioning is relatively simple. So it is assumed as a general rule that the beam-to-column and column-to-foundation connections are properly over-proportioned with respect to the bending moment ultimate capacities of the columns, following the rules 5.11.2.1.1/2 of *EC8* (see also 5.2 of *DG0*). Floor connections involved in the diaphragm action can refer to some approximate methods (see again 5.2 of *DG0*).

In any case an over-proportioning of the structural connections can be made referring to the forces obtained from a structural analysis performed with $q=1,5$.

3.2 Suggestions for the structural model

For the numerical model of the structure, the ordinary linear elements (beam type) can be used, positioned along the axis of the members. It is recommended to reproduce the different eccentricities between the members, using link rigid elements at their joints. The connections between the elements shall be faithfully represented with their degrees of freedom in the different planes. One should consider that, if the connections are modelled with no deformability (e.g.: fixed "built in" full support or hinged support), the results of the analysis could lead to very high joint forces. The actual even small deformability of the connections can lower sensibly these forces. More reliable results can be obtained if also the actual deformability of the connections is reproduced in the model.

The floor elements can be reproduced as linear elements concentrating their mechanical properties along the axis. To reach the actual points of their connections, link rigid elements can protrude from the axis. The diaphragm action of the floors shall be properly represented, implicitly by the lay-out of their members or explicitly through the options provided by the computation code.

If the wall panels are introduced as members in the model, they can be reproduced as linear elements distributing their weight along the axis. Their supports shall reproduce faithfully the isostatic arrangement of the connections. To reach the actual points of the connections, where some response parameters are needed, link rigid elements can protrude from the panel axis.

If the wall panels are introduced as masses in the model, their total mass M shall be transferred to the sustaining members in a ratio R depending on the connection arrangement.

For one storey structures with vertical panels, in the horizontal orthogonal y direction this ratio is given by

$$R_y = \frac{0{,}67Mh}{h_o}$$

where h is the height of the panel and h_o is the elevation of its upper support connected with the roof deck. In the in-plane horizontal x direction the same ratio R_x=R_y can be assumed for a pendulum support arrangement, a null R_x=0 ratio can be assumed for a cantilever arrangement with upper sliding connections.

For one storey structures with horizontal panels, in the orthogonal *y* direction their mass *M* shall be shared between the two lateral supporting columns, amplified as a function of their elevation h_i :

$$R_y = \frac{0{,}5Mh_i}{h_o}$$

where h_o is the elevation of the roof deck. In the in-plane horizontal *x* direction their mass shall be transferred, with the same amplification, to the lateral columns on the basis of the constraint degree of the corresponding support.

3.3 Rocking systems

As described in Clause 3.1 of *DGA*, the vertical panel of Figure 3.1 keeps its stability in its plane until the horizontal top force H is not greater than the limit force $H_o = \frac{Gb}{2h}$, where *G* is the weight of the panel and the geometrical quantities *b* and *h* are indicated in the quoted figure. When $H>H_o$ the panel starts rocking around its lower corner like an inverted pendulum with a restoring force H_o that for small displacements remains constant. At the reverse motion the panel seats back again on the base side and starts a new opposite cycle similar to the previous one. To catch such vibration motion a refined dynamic analysis should be applied for the solution of the non-linear algorithms inclusive of the unilateral effects of the base supports.

Considering that the pretty small value of the limit force H_o can prevent the rocking motion only for low actions, for practical design applications a simplified approach can be used, based on a linear elastic structural analysis for each of the two possible structural schemes (integrated and isostatic). The design approach can therefore develop with a first model referred to the integrated system with cantilever panels fully fixed at their base and connected with an equivalent hinge to the roof and a second model referred to the isostatic system with pendulum panels connected with two end hinges.

Starting with the integrated system, the first analysis refers to the SLS seismic action, evaluated using the pertinent elastic response spectrum. Its outcome provides the forces and displacements. If the corresponding connection forces are not greater than H_o, the calculated displacements are used for the verification of the drift limits. If they are greater, the analysis of the isostatic model is necessary.

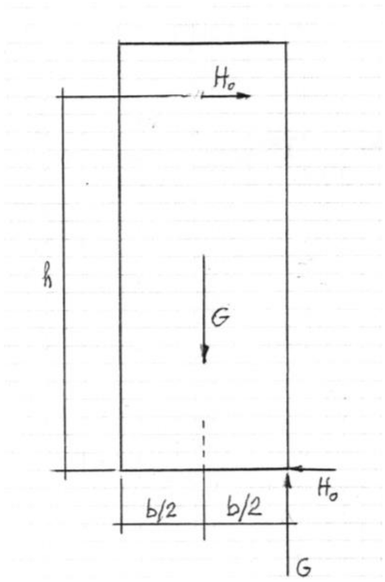


Figure 3.1

The second analysis of the integrated system refers to the ULS seismic action (no-collapse requirement), evaluated using the pertinent design response spectrum with the reduction (behaviour) factor $q=1$. Its outcome provides the forces and displacements. If the corresponding connection forces are not greater than H_o, the forces calculated in the structure are used for the strength verification. If they are greater, the analysis of the isostatic model is necessary.

When necessary, the analyses of the isostatic system are performed, neglecting the restoring force H_o. The panels participate to the response of the structure only as masses without any stiffness and can be modelled as indicated in 3.2. For SLS the elastic response spectrum is used and the resulting displacements are verified against the required drift limits. For the ULS the design response spectrum is used the q factor of the frame systems and the resulting forces are used in the strength verifications.

In any case the forces in the panel connections for their strength verification, taking into account the impulsive effects of the dynamic action, shall be taken at least equal to $2H_o$.

4. INTEGRATED SYSTEMS

For new buildings with integrated arrangements of wall panel connections, the structural analysis under seismic action shall refer to the dual wall-frame system that includes in the resisting structure columns, beams, floor elements and cladding panels with their connections.

In addition to the ordinary output data used for the verification of member resistance at ultimate limit state, the forces in the panel-beam joints shall be provided for the verification of the pertinent capacities of panel connection devices.

This chapter concerns guidelines on the design of panels with integrated connections. It is noted, however, that other types of panel connections can be used together with integrated ones, which shall be designed following the guidelines reported in the corresponding chapters. In such cases, the most unfavourable value shall be assigned to parameters referring to the overall response of the structure (e.g. the behaviour factor q).

4.1 General considerations on seismic design

4.1.1 Behaviour factor

In buildings with integrated arrangements of panel connections, the panel walls participate in the lateral load resisting system. In general, the lateral resistance of the panel walls is higher than 50% of the total lateral resistance of the building, therefore, such buildings are classified as *wall systems or wall-equivalent dual systems* according to *EC8*.

In addition, according to the Note of clause 5.11.1.3.2(3) of *EC8*, precast buildings with wall panels shall be designed for *Ductility Class Medium* (DCM). Therefore, the maximum allowed basic value of the behaviour factor is q_0 = 3.0 according to Table 5.1 of *EC8*.

Based on the above, the overall behaviour factor that will be used in the seismic design shall be calculated as follows:

$$q = q_0 \, k_w \geq 1{,}5$$

where

q_0 = 3,0

$$k_w = \frac{(1 + \alpha_0)}{3} \leq 1{,}0 \text{ but not less than } 0{,}5$$

α_0 is the prevailing aspect ratio of the panel walls, where $\alpha_0 = \dfrac{h_{wi}}{l_{wi}}$.

4.1.2 Design of wall panel connections

Due to the large stiffness of buildings with integrated panel walls, small storey displacements are expected to occur. Therefore, the prevailing energy dissipation mechanism would come from the wall panels and from the possible plastic deformation of their connections. Although common panel connections (except connections with bolted plates) possess considerable ductility (see Chapter 4 of *DGA*), it is not recommended to allow plastic deformation of the connections. This because, for large displacements, significant plastic deformation of the bars or bolts of the connecting mechanicsms occurs, leading to the slippage of the panels and to considerable pinching during the cyclic response. For this reason, the connections shall not be designed for contributing to the ductile response of the system but shall be overdesigned in the sense of clause 5.11.2.1.2 of *EC8*.

The design action-effects of the connections shall be derived on the basis of the capacity design rules. For the dual wall-frame systems considered in this chapter, the application of capacity design criteria is, in general, difficult. To overcome this difficulty, it is suggested that the over-proportioning of the structural wall connections shall be made referring to the forces coming from a structural analysis performed with a behaviour factor q=1.5

4.1.3 Design aspects

It is not needed to design the precast wall panels as ductile walls, but it is sufficient to dimension them following the design criteria of *EC8* for Lightly Reinforced Walls.

Special care should be given to the proper dimensioning and reinforcing of the regions of the panels close to the connections, where large forces develop. Also, large compressive stresses are expected to develop at the corners of the walls, due to rocking, thus adequate reinforcement shall be provided at these places. Finally, the proper anchoring of the connecting devices is of vital importance.

For structures with integrated arrangements of wall panel connections, special attention shall be addressed to the analysis of floor and roof diaphragms, verifying their elements and relative internal and peripheral connections for the transfer of the inertia forces to the lateral resisting walls. If a null diaphragm action is offered by the roof arrangement, the verification of the compatibility of the joint distortions shall be made.

It is noted that the large stiffness of the panel walls might cause the development of large forces not only to the panel connections but also to all other connections of precast members (roof-to-roof, roof-to-beam, beam-to-column), which have to be verified.

4.2 Structural modelling

4.2.1 General issues

In general, due to the large stiffness of the panel walls, the model of the structure that will be used in the analysis must reflect the real stiffness distribution within the structure, in order to be able to capture accurately the distribution of the internal forces that develop during the seismic excitation, especially the forces induced to the connections between the precast members.

For the numerical model of the structure, beam/column elements in combination with plate elements can be used, positioned along the axis or in the mid-plane of the corresponding structural element. It is recommended to reproduce the different eccentricities between the members, using link rigid elements at their joints. The connections between the elements shall be faithfully represented with their degrees of freedom in the different planes. Especially for the wall connections, one should consider that, if the connections are modelled with no deformability (e.g. fixed "built in" full support or hinged support), the results of the analysis could lead to unrealistic distribution of the joint forces. Thus, the actual deformability of the connections is deemed necessary in order to obtain reliable results. This issue is discussed in section 4.2.2.

To avoid an excessive number of modes to be considered in the modal analysis, it is recommended to neglect all the local vibrations of the elements by considering the masses, mainly the masses of wall panels, concentrated at the joints of the frame structure.

4.2.2 Modelling of the wall panels

While the use of plate elements for modelling the behaviour of the panels is a better choice from a theoretical point of view, beam-column elements can also be selected. Each panel can be modelled with 5 elastic elements (Figure 4.1): the main element is placed at the centreline of the panel while the remaining four elements are used to reach the connections with the beams.

One should consider that, if the connections are modelled with no deformability (e.g.: fixed "built in" full support or hinged support), the results of the analysis could lead to very high joint forces. The actual even small deformability of the connections can lower sensibly these forces. More reliable results can be obtained if also the actual deformability of the connections is reproduced in the model.

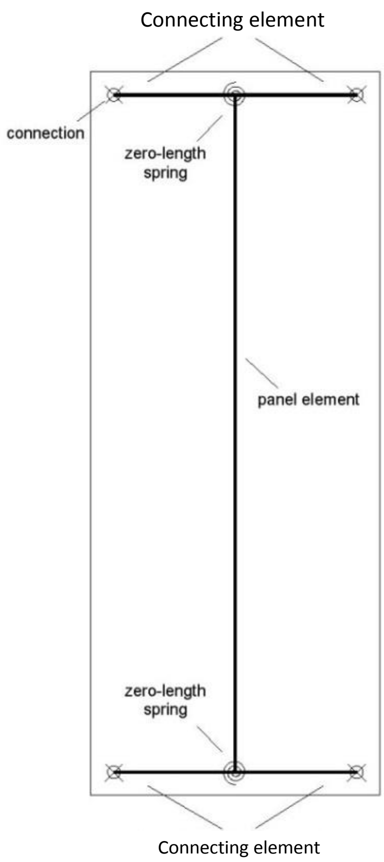


Figure 4.1

In order to account for the deformability of the connections, it is recommended that zero-length rotational springs are placed at the ends of the panel element (Figure 4.1) which capture the overall rotational response at the panel-beam joint. This response is dominated by the rocking of the panel, which leads to the tension of the connection at the uplifting side and the compression of the concrete at the opposite side of the panel. The rotational springs capture the overall moment-rotation relationship during rocking. The calculation of their stiffness is given in the ensuing.

In case that the two upper fastenings are replaced by vertically sliding connections to allow thermal expansion of the panel (see Fig. 4.2 of *DGA*), the rotation at the top side of the panel is released. Therefore the stiffness of the top rotational spring is set to zero (pinned connection between the panel element and the top connecting elements).

4.2.3 Stiffness of zero-length rotational spring

The stiffness K_θ of the zero-length rotational spring can be calculated assuming that the connection under tension can be simulated by a vertical linear spring of stiffness K_z (K_z refers to the stiffness up to the theoretical point of yielding). Let M be the bending moment at the base of the panel, which produces rotation θ (Figure 4.2). Then, the vertical displacement at the connection (elongation of the equivalent connection spring) is $d_z = s\theta$, where s is the distance of the centerline of the connection from the neutral axis O.

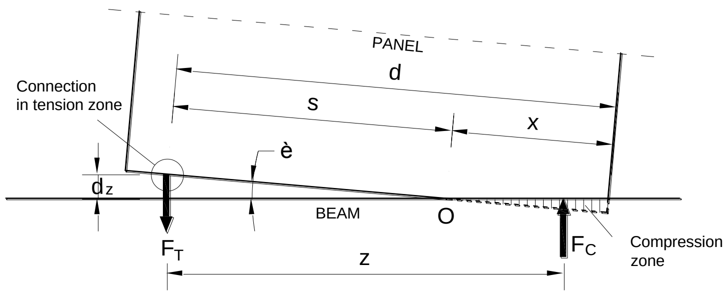


Figure 4.2

The tensile force induced to the connection is: $F_T = K_z d_z = K_z s\theta$. On the other hand, one can write: $F_T = \frac{M}{z}$, where z is the inner lever arm of the tensile and the compressive forces developed at the base of the panel. Combining the above equations and setting $K_\theta = \frac{M}{\theta}$, the following relation can be derived:

$$K_\theta = K_z z\, s$$

The values of z and x can be estimated following standard approximations, usually made for reinforced concrete sections: the inner lever arm z can be set equal to z=0.9d, where d is the distance of the centreline of the connection from the opposite edge (effective depth); and the distance s can be calculated as: s=d–x, where x is the length of the compression zone that can be approximated by x=0.25d. Thus, s=0.75d.

In what concerns the value of K_z, for connections with protruding bars and wall shoes (see Chapter 4 of *DGA*) one can write:

$$K_z = \frac{EA}{L_{eff}}$$

where:

E is the modulus of elasticity of the steel of the bars/bolts;

A is the stressed area of the bars/bolts; and

L_{eff} is the equivalent length of the spring, denoting the effective length in which the elongation of the bars/bolts takes place.

Based on the data obtained from the experimental investigation of the connections, L_{eff} can be estimated by:

$L_{eff} \cong 15Ø$

where Ø is the diameter of the bar or the bolt. It is noted, however, that, due to the limited number of the available experimental data, this approximation needs further verification by additional experimental and numerical investigations. If the above equation for K_q is used, it is suggested that two analyses shall be performed: one with double the value of K_q, from which the maximum forces in the connections will be determined, and one with half the value of K_q, from which the maximum displacements will be determined.

For connections with bolted plates, the calculation of the proper value of K_z is not easy, since it is affected by a number of factors which cannot be easily modelled, as the shear deformation of the bolts, the elongation of the steel plate, the distortion of the holes and the plate itself, etc.

4.2.4 Pre-dimensioning of panel wall connections

In order to calculate the rotational stiffness K_θ of the panel models by applying the equations above, the cross section of the bars/bolts is needed. Therefore, an initial evaluation of the forces expected to develop in the connections is necessary. This pre-dimensioning of the connections can be based on simplified assumptions concerning the distribution of the lateral forces, as the ones reported in the following.

It must be emphasized that the following analysis gives an estimation of the average forces expected to develop in the panel connections. Rigorous analytical investigations have shown that the forces induced to panel wall connections might change significantly from place to place, depending on the position of each panel in the load-resisting structural system. Therefore, this analysis can only be used for the pre-dimensioning of the connections, while the final verification of the connections shall be based on the actual forces derived from the dynamic modal analysis. In case that this verification shows that some connections need to be modified, the analysis shall be repeated.

Panels with four connections

Let us assume that there are n vertical panels at each side of the building along the direction of the seismic action and that each panel is pinned to the top and the bottom beam by two connectors at each edge. Each panel has dimensions $L_{panel} \times H_{panel}$, while L and H are the horizontal and the vertical distance between the connections of the panel (Figure 4.3). Then, one can define the coefficient C_1 as

$$C_1 = L / L_{panel}$$

In general, the total length L_{tot} of the building sides is not fully covered with panels, thus the coefficient C_2 can be defined as

$$C_2 = n \cdot L_{panel} / L_{tot}$$

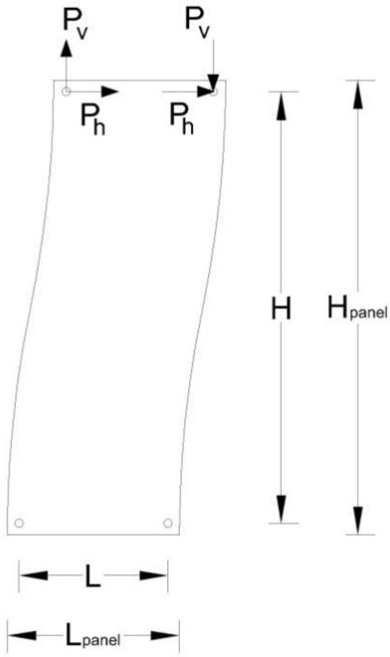


Figure 4.3

Note that C_2=1 for sides fully covered with panels, but it can be significantly smaller than unity for sides with long openings (partially covered).

For a symmetrical building and cladding wall panels placed at the two external sides, and accounting for the large stiffness of the panels compared with the stiffness of the precast frame, it can be assumed, as a first approximation, that the base shear due to the earthquake load, P_{base}, is taken only by the panels. Then it can be proved that the horizontal force, $P_{i,h}$, and the vertical force, $P_{i,v}$ that are induced to each panel connection are (see Figure 4.3):

$$P_{i,h} \cong \frac{P_{base}/2}{2n} \qquad P_{i,v} = P_{i,h}\frac{H}{L}$$

The total force induced to each connection is $P_i = \sqrt{P_{i,h}^2 + P_{i,v}^2}$ and using the above relations one can write:

$$\frac{P_i}{P_{base}} = \frac{1}{4}\sqrt{\frac{1}{n^2} + \left(\frac{H}{C_1 C_2 L_{tot}}\right)^2}$$

In general, for values of n larger than about 4, the term $1/n^2$ is much smaller than the term $\left(\frac{H}{C_1 C_2 L_{tot}}\right)^2$ and can be neglected in Equation (3.9). Denoting with $P*$ the base shear per unit length, i.e.

$$P* = \frac{P_{base}}{L_{tot}} = \frac{C_1 C_2 P_{base}}{nL}$$

one obtains:

$$P_i = \frac{P*}{C_1 C_2} \cdot \frac{H}{4}$$

Panels with vertically sliding top connections

For the same assumptions with the above analysis, but considering panels free to rotate at their top, it can easily be proved that

$$P_i = \frac{P*}{C_1 C_2} \cdot \frac{H}{2}$$

showing that the forces induced to the bottom connectors are double than the ones for panels with four connections.

Remarks

The equations above imply that the force induced to each connection is independent of the width of the panels. This practically means that the forces at the wall connections cannot be reduced by using more panels of smaller length or less panels of larger length. However, the connection forces greatly depend on the "*coverage*" of the external sides by panels (coefficient C_2) and increase significantly in case of sides with long openings (partially covered by panels).

The force induced to each connection is linearly increasing with the vertical distance H of the connections, i.e. with the height of the storey.

The major component of P_i is in the vertical direction.

4.3 Wall panels detailing

Generally, in the current practice, the detailing of precast cladding panels does not fulfil the code requirements for structural shear walls, especially in what concerns their minimum thickness. In order to guarantee thermal insulation, common solutions are: (a) sandwich wall made of two lateral thin concrete layers interconnected by metallic devices like steel lattice girders, with interposed insulating material; (b) single thin concrete layer with stiffening ribs plus an insulating layer of non-structural material attached at the opposite side. Both solutions are insufficient to resist the large forces induced to them during strong ground shaking in integrated system, where areas of significant dimensions made of massif concrete are needed for the proper anchoring of the fastening devices and the transfer of the large forces that develop.

Specific requirements for integrated cladding panels used in the dual wall-frame systems are presented here-after. These requirements are formulated through proper adaptation of the *EC8* rules for the cast-in-situ shear walls and aim to ensure strong fastening of the connectors, adequate in plane shear resistance and sufficient ductility. Specifically:

- Panels shall have a solid bearing layer of at least 150 mm of thickness;
- A double reinforcing mesh of ductile steel shall be provided at the two faces;
- In both directions, the sides of the mesh shall be not larger than 200 mm and the bars should have diameter at least 8 mm;
- A perimeter reinforcement shall be added with at least 2 longitudinal bars of diameter Ø ≥ 12 mm and edge links of diameter Ø ≥ 8 mm;
- Proper anchoring reinforcement of the inserts shall be located at the connection points.

Panels with openings shall be properly designed for the transmission of the expected in plane actions through the lateral posts of the openings. Proper reinforcement, specifically continuous steel ties, horizontal or vertical, should be provided around the openings, similar to the reinforcement placed around openings in ductile shear walls. As a minimum, these ties should satisfy clause 9.10 of *EC2*.

5. DISSIPATIVE SYSTEMS

The use of dissipative connections, placed between the panels as described in Chapter 5 of *DGA*, introduces a source of friction or plastic hysteretic dissipation of energy in the dual wall-frame structural system. The contribution to energy dissipation of these connections in seismic behaviour of the overall structural system depends on the magnitude of their deformation and force capacities with respect to those of the overall structural system in which they are inserted.

The friction dissipative devices considered in this document refer to vertical panels. They have a very small initial elastic flexibility and a friction slide play limited to few centimetres, with a constitutive law that can be represented by a rigid-friction diagram (see 5.2.3 of *DGA*). For the corresponding limited floor drifts the columns that work in parallel, remain usually within the elastic field and the dissipation of energy comes only from the dissipative devices. Therefore, with respect to the high seismic response of the initial stiff dual wall-frame system with fixed connections, the force reduction effects can come only from the set of dissipative devices interposed between the panels when the slip threshold is overcome. To obtain a sensible force reduction, a suitable quantity of energy shall be dissipated. In the meantime the stiffening effect that allows to reduce sensibly the displacements with respect to those of the bare frame comes from the total contribution given by the friction devices in terms of resisting force. The same considerations are valid for the multi-slit devices.

The steel cushions described in Clause 5.4 of *DGA* have a larger initial elastic flexibility that can modify the vibration properties of the stiff dual wall-frame system, moving them towards the properties of the flexible frame. Also the subsequent plastic slide play is larger, so that a certain contemporary contribution to energy dissipation can come from both cushions and columns.

For the panels used in the dissipative systems the same detailing rules of 4.3 shall be applied.

5.1 General indications on seismic design

The general approach for the design of precast structures with dissipative systems of connections should be based on a non linear dynamic analysis applied to the spatial model of the dual wall-frame structure where the mutual panel connections are represented by their proper constitutive laws as presented in Chapter 5 of *DGA*. For the details of the spatial model, reference can be made to Points 3.2 and 4.2.

A linear modal dynamic analysis of the dual wall-frame structure can be performed with the proper behaviour factor q representing the force reduction due to the dissipative connections. For the calibration of this behaviour factor, that is not presently regulated by the pertinent design codes, a preliminary parametric investigation is needed, comparing with a probabilistic approach the results obtained by the non linear and linear dynamic analysis for a significant set of structural situations.

Specific application guidelines are added in Clauses 5.2 for friction devices, while in Clause 5.3 an alternative approach is presented for steel cushions.

In terms of roof drift d_x of an one-storey building, the relative slide play $\pm s_z$ between two adjacent panels (see Figure 5.1) leads to $d_x = \pm \frac{s_z h}{b}$ that, for the common dimensions of the panels, corresponds to about three times its value.

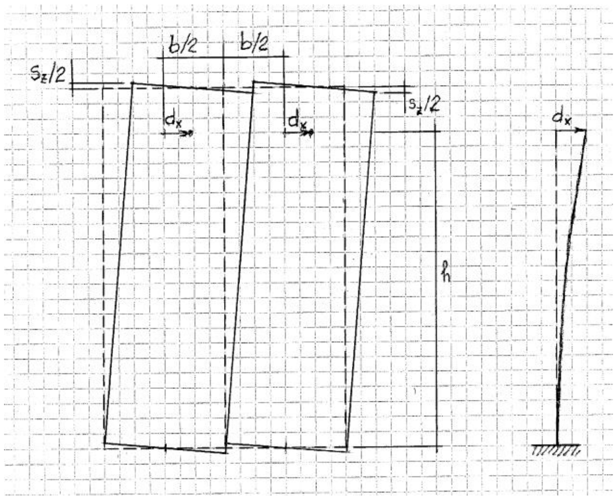


Figure 5.1

Figure 5.2 shows the forces transmitted between the panels and to the frame structure, where (a) is the end panel that transmits a relevant vertical force to the foundation, (b) is an internal panel that exchanges vertical shear forces at both sides, (c) is a column of the frame connected at the top with the same roof diaphragm.

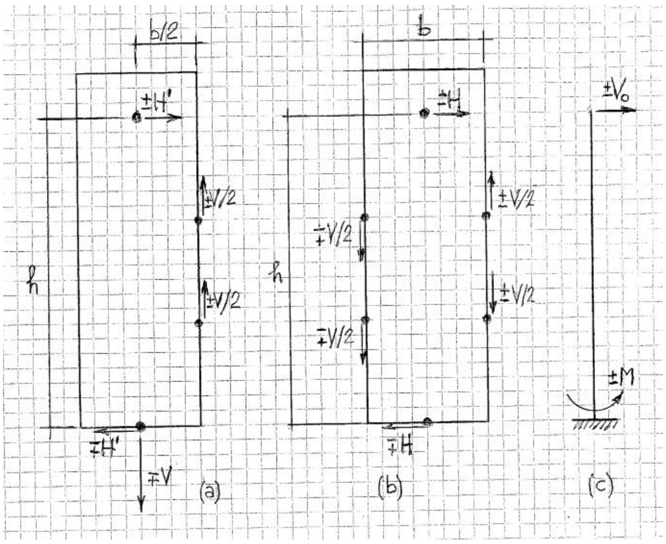


Figure 5.2

The horizontal resistance contributions can be deduced from the rotation equilibrium:

$$H' = \frac{Vb}{2h} \qquad H = \frac{Vb}{h} \qquad V_o = \frac{M}{h}$$

If V is the threshold force of an elastic-plastic model of a dissipative device and M is the contemporary moment of the column base, these horizontal forces correspond to the maximum response of the structure. Calling F_p the sum of the contributions H' and H of all the panels and F_c the sum of the contributions V_o of all the columns, this maximum response is

$$F = F_p + F_c$$

where one can assume F_c=0 if the storey drift is kept within small values. Compared to the maximum response F_{max} of the integrated dual wall-frame system, where the initial high stiffness is given by the large walls with fixed panel connections, this response leads to the required reduction (behaviour) factor

$$q = \frac{F_{\max}}{F}$$

The calculation of F_{max} can be referred to the total vibrating mass of the building and to the maximum spectral response of the site.

The equation above can be used to proportion the dissipative devices in number and strength assuming a proper value of the q factor. For the panels used in the dissipative systems the same detailing rules of 4.3 shall be applied. It holds as long as the total force contribution of the dissipative connections under the design seismic action is much higher than the total contribution of columns and the corresponding ultimate floor drift is compatible with the maximum displacement capacity of the dissipative devices.

5.2 Structures with friction devices

The current methods of linear static analysis and modal analysis with response spectrum rely on the possibility to identify a proper force reduction (behaviour) factor depending on the dissipative capacity of the earthquake resisting system. For precast structures with dissipative connections of the cladding panels, the behaviour factor has still to be calibrated and validated. For this reason, the seismic design of this kind of system should be based on proper nonlinear time-history dynamic analyses under prescribed ground motions. However, this approach is computationally expensive and not handy in engineering practice. As an alternative, the classical linear methods of seismic analysis could be applied based on a conservative estimation of the behaviour factor.

With this regard, it is noted that recent experimental and numerical investigations demonstrated that precast structures with dissipative panel connections exhibit large ductility and dissipation capacity, as shown in Figures 5.3 and 5.4 for a full scale prototype of precast structure submitted respectively to pseudodynamic and cyclic tests at ELSA Laboratory in the scope of Safecladding Project. Based on these results, that show the high ductility capacity of the system, it is suggested to adopt the behaviour factor of concrete frames as conservative estimation of the behaviour factor of the combined frame-panel earthquake resisting system.

Based on this assumption, a simplified seismic analysis could be carried out as follows:

1. definition of the elastic response spectrum of the site;
2. definition of the design response spectrum based on the conservative estimation of the reference value of the behaviour factor q;
3. static analysis or dynamic modal analysis of the 3D model of the dual wall-frame structure with integrated arrangement of the panels, attached to each other with fixed connections (it is worth noting that the first natural vibration period of the frame-panel earthquake resisting system is small and falls in the plateau region of the spectrum);
4. evaluation (from the above analysis) of the forces in the panel-to-panel connections and consequent design of the dissipative devices;
5. capacity design of the panel-to-frame connections according to Figure 5.2 and related equations where $V=V_{max}$ of 5.2 of *DGA*;
6. evaluation of the inelastic maximum top displacement of the dual wall-frame structure associated with the reference value of the behaviour factor (elastic displacement multiplied by $(1+q)/2$) and computation of the corresponding inelastic relative displacements in the dissipative panel-to-panel devices, to be compared with their kinematic capacity δ_{max}.

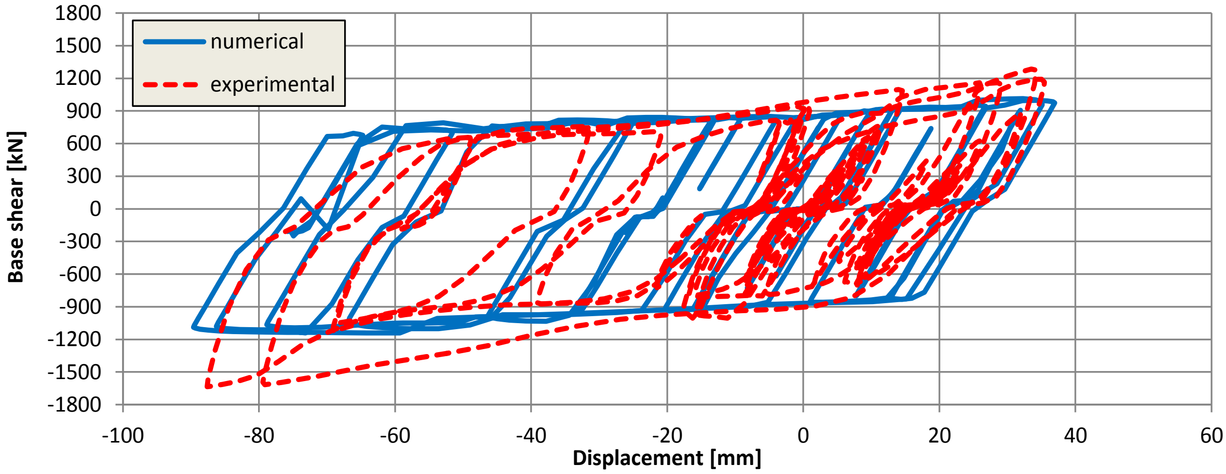


Figure 5.3

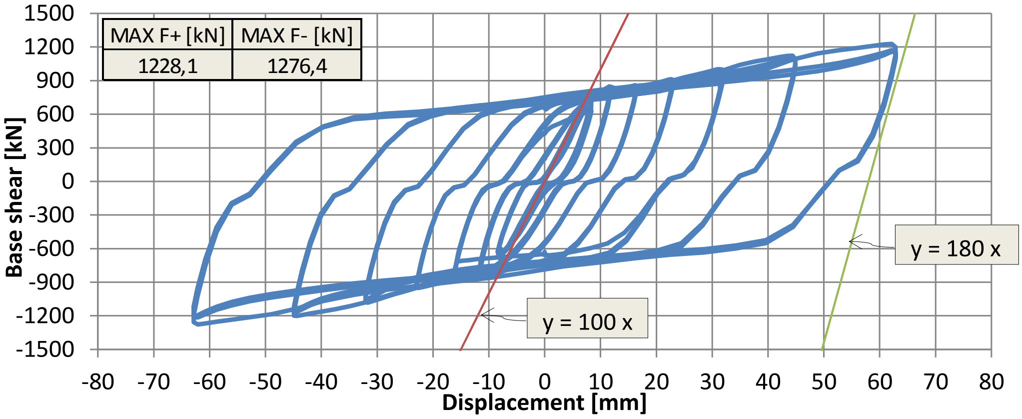


Figure 5.4

5.3 Structures with steel cushions

The steel cushions are elements with very large displacement capacity. Their energy dissipation capability is function of the top displacement or drifts of the structure. A combined energy dissipation of RC prefabricated columns and steel cushions is the type of behaviour that is desired. In high levels of displacements, the cushions will contribute largely to the overall energy dissipation of the entire structure. The key points here are:

- target displacement of the system
- ductility demand on the cushions and on the columns
- amount of hysteretic energy and the overall damping

These characteristics of the cushions suggest the use of a displacement-based design procedure rather than a forced-based one because the displacement-based procedures give the freedom to the designer to select the desired base shear contribution of the cushions at the beginning of the design process.

In a summary review of the fundamentals of Direct-Displacement Based Design (DDBD), initially the design displacement at maximum response δ is determined and the corresponding equivalent viscous damping ε is estimated from the expected ductility demand at the limit state of interest. It is noted that the equivalent damping is representative of the combined elastic damping and the hysteretic energy absorbed during inelastic response. The effective period T_e at maximum displacement response can be read then from a set of displacement for different levels of damping. Consecutively, the effective (secant) stiffness K_e of the equivalent SDOF system at maximum displacement can be easily computed by the known equation for the period of a SDOF oscillator.

In order to estimate the target displacement, the procedure below should be followed:

1. The fraction of the lateral load, β, to be carried by the cushions, is assigned by the designer. This β value should be in the range of 20 to 60%, depending on the stiffness of the rigid diaphragm.

2. Define the design displacement for the columns by calculating the yield and the ultimate displacements. The ultimate drift will be dictated either by the stability-related drifts of the columns, such as the maximum allowable rotation to prevent the toppling of the beams, or by the material strain limits of the column.

   First, the yield displacement of the frame, $\Delta_{y,c}$, will be calculated as given below:

   $$\Delta_{y,c} = \frac{\phi_y \left(H + L_{sp}\right)^2}{3}$$

   where ϕ_y is the yield curvature and is calculated as given below:

   $$\phi_y = \frac{2{,}10\,\varepsilon_y}{h_c}$$

   Consequently, the overall design displacement of the frame, Δd,f, is estimated as given below and compared to the displacement limit required for keeping the stability. The minimum of the two displacements is assumed as the design displacement of the frame.

   $$\Delta_{d,f} = \Delta_{y,c} + \Delta_{p,c} = \frac{\phi_y \left(H + L_{sp}\right)^2}{3} + \left(\phi_{ls} - \phi_y\right) L_p H$$

3. Assume a yield displacement for the cushions to be used. This yield displacement will be the first assumption since the steel cushions are not chosen yet, but the yield displacement of the cushions is highly correlated with their geometric properties, thus an accurate assumption for the yield displacement of the cushions can be made initially and can be revised with a single iteration at the end of the design process, before even starting the computer modelling.

4. Calculate the ductility demand of the RC columns and of the steel cushions.

5. Estimate the hysteretic overall damping for the structure (3rd of the equations below) from the contributions of the columns (1st of the equations below) and of the steel cushions (2nd of the equations below), by using the equations below. Note that the formula for the RC columns is based on Takeda-like hysteretic responses, while the formula for the cushions is based on bilinear behaviour. The elastic damping for the precast RC columns is assumed 5%. If this level of elastic damping is deemed to be high and a lower value of say 2% is to be adopted, then the coefficients in the following expressions need to be revised.

6. $$\xi_f = 0{,}05 + \frac{0{,}565(\mu - 1)}{\mu\,\pi}$$

   $$\xi_c = 0{,}05 + \frac{0{,}519(\mu - 1)}{\mu\,\pi}$$

   $$\xi_{str} = \xi_f\,(1 - \beta) + \xi_c\,\beta$$

7. Reduce the design displacement spectral values by using the formula below:

$$\eta = \sqrt{\frac{7}{2+\xi}}$$

8. For the design displacement and by using the over-damped design displacement spectrum, calculate the effective period, T_e
9. Calculate the effective stiffness, K_e:

$$K_e = \frac{4\pi^2 m_e}{T_e^2}$$

10. Calculate the design base shear of the structure, V_B

$$K_B = K_e \Delta_d$$

11. Distribute the design base shear to the cushions and to the frame by using the initially assumed β value.
12. Check the required design shear versus initially assumed yield displacement for the cushions and select a compatible cushion type. If not available, select the closest and conduct only one iteration between steps 3 to 10.
13. Conduct a standard reinforced concrete design procedure per *EC2* for the columns.

ANNEX 0 - PARAMETRIC ANALYSES ON BUILDING TYPOLOGIES

In order to verify the actual influence of cladding panels on the seismic behaviour of precast buildings, a wide parametric investigation has been performed by means of numerical analyses on several structural assemblies representing the most common typologies of precast buildings. The aim is to analyse how the different types of connection systems (isostatic, integrated and dissipative) play their role in the structural response, so to spot the situations where this role is important and evaluate the conditions for practical applications. For this systematic parametric investigation, frame systems of structures for one-storey buildings with industrial and commercial destination have been considered.

Dynamic elastic (ELD) analyses have been performed with reference to Serviceability Limit State and dynamic non-linear analyses (NLD) have been performed with reference to No-Collapse Limit State. As input action a modified Tolmezzo accelerogram has been used (signal registered in Tolmezzo on 1976 supplemented in frequencies to make it wherever compatible with *EC8* spectrum for subsoil B). The analyses have been elaborated with a 3D overall model of the structure. The panel connections have been represented with the simplified assumption of "totally free to move" or "totally fixed" or "elastic-plastic" respectively for the isostatic, the integrated and the dissipative system.

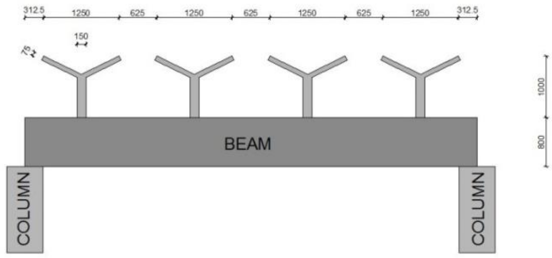
The following construction parameters have been assumed in the parametric analyses:

- *structural arrangement*: regular
- *roof deck*: short beams with long roof elements
- *structure height*: 7,5 m
- *cladding walls*: on four sides
- *type of panels*: vertical
- *panel connections* (isostatic - integrated - dissipative systems)
- *shape ratio*: elongated 3/1 - medium 3/2 - compact 3/3
- *roof diaphragm*: null - deformable - rigid
- *action intensity*: 0,18g - 0,36g - 0,60g
- *action direction*: longitudinal - transversal

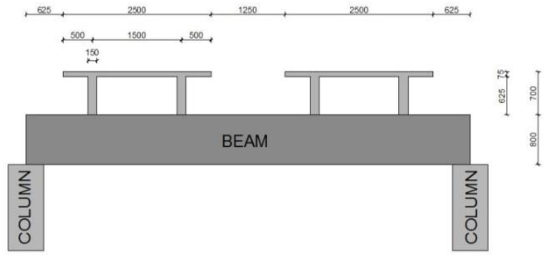
Details on the building typologies and on different analyses performed are given hereafter. A proportioning of sizes and reinforcement has been made following *EC8* design rules, with PGA=0,30g and subsoil B. For the columns a concrete Class C45/55 is adopted together with a steel Class B450C. The calculation, referred to the bare frame structure of the three shape ratios with an isostatic system of vertical panel connections, is reported in Annex A. The numerical analyses have been performed with the same materials and dimensions, assuming in particular for the non-linear dynamic analysis the mean values of the strengths.

The combination of all the aforementioned parameters leads to 3x3x3x3x2=162 different cases to be analysed and this requires suitable criteria for achieving an effective and synthetic representation of the results. A reduced number of output parameters are identified as significantly representative of the structural response under earthquake, as a function of the investigated typology case. The list is specified below.

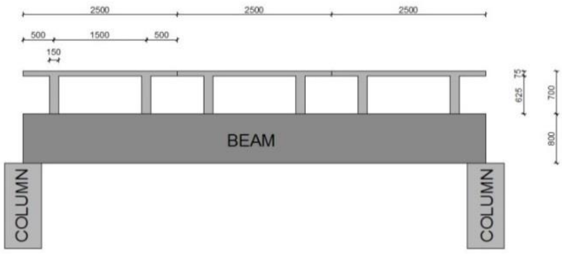
In Figure 1 the three type of roof diaphragm are shown as actually available in the ordinary production typologies: (a) spaced Y-shape roof elements with single rib end connections for a null diaphragm action; (b) spaced double-Ts elements with double rib end connections for a deformable diaphragm; (c) attached double-Ts connected to each other for a rigid diaphragm.



(a)



(b)



(c)

Figure 1

In Figure 2 the plans and the sections of the three 3/1, 3/2 and 3/3 shape ratios are shown (only for the rigid roof diaphragm): the lay-out consists of 1, 2 or 3 roof bays of 20,0 m in x direction and 8 beam spans of 7,5 m in y direction. The height is 7,5 m.

Table 1 summarizes the 162 different scheduled analyses. Table 2 shows the "type table" in which the calculated numerical values of the parameters are introduced. The definitions of the parameters are listed below. The analyses for isostatic connections have been repeated two times (for vertical and horizontal panels). The analyses for integrated connections have been repeated two times (for 3 and 4 connections per panel). The analyses for dissipative connections have been repeated two times (for plastic and friction devices). In the following pages 6x9=54 tables are reported with the pertinent parameters. Possible specifications are added in foot-notes joined to the tables.

The 6 sets of 9 tables refer respectively to isostatic with vertical panels, isostatic with horizontal panels, integrated with 3 joints, integrated with 4 joints, plastic dissipative and friction dissipative systems of connections and have been elaborated respectively by the research groups of Ljubljana University, National Technical University of Athens, Istanbul Techical University and Politecnico di Milano. For the same layouts of Figures 1 and 2, different numerical models have been set-up by the groups, with some different options provided by the calculation codes used. These differences lead to out-put data that are not perfectly comparable between the different sets of tables.

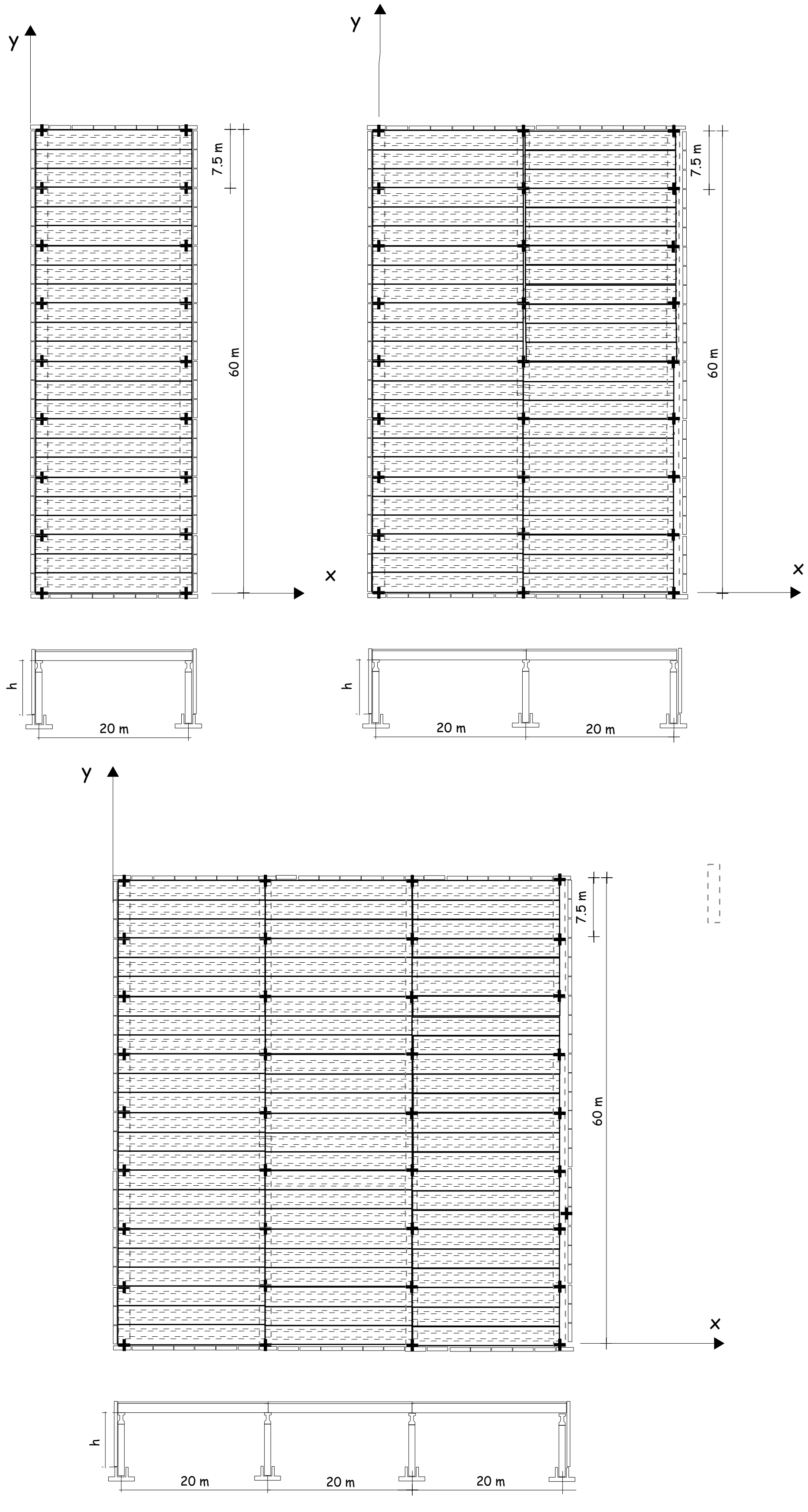


Figure 2

ONE-STOREY REGULAR – 4 SIDES VERTICAL CLADS - h= 7,5 m				
code	**system**	**shape**	**diaphragm**	**direction / *PGA***
iso3/1NULx	SOSTATIC	3:1	Null	x / *0,18-0,36-0,60*
iso3/1NULy				y / *0,18-0,36-0,60*
iso3/1DEFx			Deformable	x / *0,18-0,36-0,60*
iso3/1DEFy				y / *0,18-0,36-0,60*
iso3/1RIGx			Rigid	x / *0,18-0,36-0,60*
iso3/1RIGy				y / *0,18-0,36-0,60*
iso3/2NULx		3:2	Null	x / *0,18-0,36-0,60*
iso3/2NULy				y / *0,18-0,36-0,60*
iso3/2DEFx			Deformable	x / *0,18-0,36-0,60*
iso3/2DEFy				y / *0,18-0,36-0,60*
iso3/2RIGx			Rigid	x / *0,18-0,36-0,60*
iso3/2RIGy				y / *0,18-0,36-0,60*
iso3/3NULx		3:3	Null	x / *0,18-0,36-0,60*
iso3/3NULy				y / *0,18-0,36-0,60*
iso3/3DEFx			Deformable	x / *0,18-0,36-0,60*
iso3/3DEFy				y / *0,18-0,36-0,60*
iso3/3RIGx			Rigid	x / *0,18-0,36-0,60*
iso3/3RIGy				y / *0,18-0,36-0,60*
int3/1NULx	INTEGRATED	3:1	Null	x / *0,18-0,36-0,60*
int3/1NULy				y / *0,18-0,36-0,60*
int3/1DEFx			Deformable	x / *0,18-0,36-0,60*
int3/1DEFy				y / *0,18-0,36-0,60*
int3/1RIGx			Rigid	x / *0,18-0,36-0,60*
int3/1RIGy				y / *0,18-0,36-0,60*
int3/2NULx		3:2	Null	x / *0,18-0,36-0,60*
int3/2NULy				y / *0,18-0,36-0,60*
int3/2DEFx			Deformable	x / *0,18-0,36-0,60*
int3/2DEFy				y / *0,18-0,36-0,60*
int3/2RIGx			Rigid	x / *0,18-0,36-0,60*
int3/2RIGy				y / *0,18-0,36-0,60*
int3/3NULx		3:3	Null	x / *0,18-0,36-0,60*
int3/3NULy				y / *0,18-0,36-0,60*
int3/3DEFx			Deformable	x / *0,18-0,36-0,60*
int3/3DEFy				y / *0,18-0,36-0,60*
int3/3RIGx			Rigid	x / *0,18-0,36-0,60*
int3/3RIGy				y / *0,18-0,36-0,60*
dis3/1NULx	DISSIPATIVE	3:1	Null	x / *0,18-0,36-0,60*
dis3/1NULy				y / *0,18-0,36-0,60*
dis3/1DEFx			Deformable	x / *0,18-0,36-0,60*
dis3/1DEFy				y / *0,18-0,36-0,60*
dis3/1RIGx			Rigid	x / *0,18-0,36-0,60*
dis3/1RIGy				y / *0,18-0,36-0,60*
dis3/2NULx		3:2	Null	x / *0,18-0,36-0,60*
dis3/2NULy				y / *0,18-0,36-0,60*
dis3/2DEFx			Deformable	x / *0,18-0,36-0,60*
dis3/2DEFy				y / *0,18-0,36-0,60*
dis3/2RIGx			Rigid	x / *0,18-0,36-0,60*
dis3/2RIGy				y / *0,18-0,36-0,60*
dis3/3NULx		3:3	Null	x / *0,18-0,36-0,60*
dis3/3NULy				y / *0,18-0,36-0,60*
dis3/3DEFx			Deformable	x / *0,18-0,36-0,60*
dis3/3DEFy				y / *0,18-0,36-0,60*
dis3/3RIGx			Rigid	x / *0,18-0,36-0,60*
dis3/3RIGy				y / *0,18-0,36-0,60*

Table 1

	ISOSTATIC/INTEGRATED/DISSIPATIVE CONNECTION SYSTEM – NULL/DEFORMABLE/RIGID DIAPHRAGM						
code	**parameters**	**X - direction**			**Y - direction**		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)						
b1 *b2*	Differential top drift (mm) Ratio (%)						
c1 *c2*	Maximum top drift (mm) Relative ( )						
d1 *d2*	^Max connection slide (mm) Relative ( )						
e1 *e2*	Max force roof-roof (kN) Relative ( )						
f1 *f2*	Max force roof-beam (kN) Relative ( )						
g1 *g2*	Max force beam-column (kN) Relative ( )						
h1 *h2*	Max force wall-structure (kN) Relative ( )						
i1 *i2*	Max force wall-wall (kN) Relative ( )						
j1 *j2*	Total base shear (kN) Relative ( )						
k1 *k2*	Total column shear (kN) Relative ()						
l1 *l2*	Mean column shear (kN) Relative ( )						
m1 *m2*	Max column shear (kN) Relative ()						

^Specify which one

Table 2

More complete sets of data can be found in the following documents:

Safecladding Project – Deliverable 2.3 – Updates on numerical and experimental analyses (Isostatic systems), February 2015

Safecladding Project – Deliverable 2.3 – Updates on numerical and experimental analyses (Integrated systems), February 2015

Safecladding Project – Deliverable 4.2 – Updates on numerical and experimental analyses (Dissipative systems), February 2015

Definitions

a1 maximum top x (or y) drift (mm)

maximum top x (or y) displacement

a2 ratio (%)

maximum top x (or y) displacement divided by the column height

b1 differential top x (or y) drift (mm)

maximum minus minimum top x (or y) contemporary displacement

b2 ratio (%)

maximum minus minimum top x (or y) contemporary displacements divided by the column height

c1 maximum top x (or y) drift (mm)

maximum top x (or y) displacement (the same of a1)

c2 relative ( )

maximum top x (or y) displacement divided by the reference x (or y) displacement

d1 maximum connection slide (mm)

maximum displacement of the considered sliding connections

d2 relative ( )

maximum displacement of the sliding connections divided by the reference x (or y) displacement

e1 maximum force roof-roof (kN)

maximum force in roof-to-roof connection

e2 relative ( )

maximum force in roof-to-roof connection divided by the reference x (or y) column shear

f1 maximum force roof-beam (kN)

maximum force in roof-to-beam connection

f2 relative ( )

maximum force in roof-to-beam connection divided by the reference x (or y) column shear

g1 maximum force beam-column (kN)

maximum force in beam-to-column connection

g2 relative ( )

maximum force in beam-to-column connection divided by the reference x (or y) column shear

h1 maximum force wall-structure (kN)

maximum force in wall panel-to-structure connection

h2 relative ( )

max. force in wall panel-to-structure connection divided by the reference x (or y) column shear

i1 maximum force wall-wall (kN)

maximum force in wall panel-to-panel connection

i2 relative ( )

maximum force in wall panel-to-panel connection divided by the reference x (or y) column shear

j1 total base shear (kN)

maximum sum of contemporary base x (or y) shear of columns and wall panels

j2 relative ( )

total base x (or y) shear divided by the reference total x (or y) base shear

k1 total column base shear (kN)

maximum sum of contemporary base x (or y) shears of columns

k2 relative ( )

sum of base x (or y) shear of columns divided by the total columns+panels base x (or y) shear

l1 mean column shear (kN)

total column base x (or y) shear of columns divided by the number of columns

l2 relative ( )

mean column base x (or y) shear divided by the reference x (or y) column base shear

m1 maximum column shear (kN)

maximum base x (or y) shear in a column

m2 relative ( )

maximum base x (or y) shear in a column divided by the mean column base x (or y) shear

Reference values

Reference x (or y) displacement

is the top x (or y) displacement calculated for a given structural arrangement, roof deck and shape ratio, assuming cladding wall panels on four sides, isostatic connection system and a rigid roof diaphragm.

Reference x (or y) column base shear

is the mean x (or y) column shear of columns calculated for a given structural arrangement, roof deck and shape ratio, assuming cladding wall panels on four sides, isostatic connection system and a rigid roof diaphragm.

Reference x (or y) total base shear

is the maximum sum of contemporary base x (or y) shear of columns and wall panels calculated for a given structural arrangement, roof deck and shape ratio, assuming cladding wall panels on four sides, isostatic connection system and a rigid roof diaphragm.

The reference values defined above are reported in Table 3.

Building type	Quantity	x-direction			y-direction		
		0.18 g	**0.36 g**	**0.60 g**	**0.18 g**	**0.36 g**	**0.60 g**
Single-bay	Reference displacement [mm]	100	191	230	96	183	236
	Reference column base shear [kN]	73	94	97	71	93	96
	Reference total base shear [kN]	1453	1886	1939	1419	1865	1910
Two-bays	Reference displacement [mm]	98	186	229	98	186	227
	Reference column base shear [kN]	75	97	99	75	96	98
	Reference total base shear [kN]	2316	2996	3074	2337	2973	3045
Three-bays	Reference displacement [mm]	104	194	238	104	196	275
	Reference column base shear [kN]	73	92	98	77	96	98
	Reference total base shear [kN]	3059	3881	4136	3239	4012	4111

Table 3

Tables of output parameters

The following pages contain 45 tables of results:

- 19 tables for the isostatic connection system: 9 with vertical panels and 9 with horizontal panels;
- 18 tables for the integrated connection systems: 9 with 3 connections per panel (2 on the foundation beam, 1 on the roof beam) and 9 with 4 connection per panel (2 on the foundation beam, 2 on the roof beam);
- 9 tables for the plastic dissipative connections between the panels;
- 9 tables for the friction dissipative connections between the panels.

Any set of tables is completed with the pertinent comments on the specific resulting structural behaviour.

MULTISTOREY PRECAST BUILDINGS

In Annex B and Annex C the structural analyses of a 3 storeys precast building are reported respectively for an integrated and an isostatic panel connection system.

Vertical panels - 1-bay building - isostatic connections - null diaphragm

	ISO3/1NUL	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
		y direction	x direction	y direction	x direction	y direction	x direction
a1	Maximum top drift (mm)	100	115	186	261	239	394
a2	Ratio (%)	1.3	1.5	2.5	3.5	3.2	5.3
b1	Differential top drift (mm)	0	113	0	243	0	383
b2	Ratio (%)	0.0	1.5	0.0	3.2	0.0	5.1
c1	Maximum top drift (mm)	100	115	186	261	239	394
c2	Relative ( )	1.04	1.15	1.02	1.37	1.01	1.71
d1	Max connection slide (mm)	100	50	186	80	239	120
d2	Relative ( )	1.04	0.5	1.02	0.42	1.01	0.52
f1	Max force roof-beam (kN)	43	58	53	70	54	104
f2	Relative ( )	0.61	0.79	0,57	0.74	0,56	1.07
g1	Max force beam-column (kN)	77	85	96	101	98	104
g2	Relative ( )	1.08	1.16	1.03	1.07	1.02	1.07
k1	Total column shear (kN)	1468	1235	1866	1458	1908	1819
k2	Relative ( )	1.03	0.85	1.00	0.77	1.00	0.94
l1	Mean column shear (kN)	73	62	93	73	95	91
l2	Relative ( )	1.03	0.85	1.00	0.78	0.99	0.94
m1	Max column shear (kN)	77	85	96	101	98	104
m2	Relative( )	1.05	1.37	1.03	1.38	1.03	1.14

Table 4a

Horizontal panels - 1-bay building - isostatic connections - null diaphragm

	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
	y direction	**x direction**	**y direction**	**x direction**	**y direction**	**x direction**
Maximum top drift (mm)	94	106	182	197	234	288
Ratio (%)	1.3	1.4	2.4	2.6	3.1	3.8
Differential top drift (mm)	0	64	0	121	0	140
Ratio (%)	0.0	8.5	0.0	16.2	0.0	18.7
Maximum top drift (mm)	94	106	182	197	234	288
Relative ( )	1.0	1.1	1.0	1.1	1.0	1.2
Max connection slide (mm)	33	33	61	36	75	61
Relative ( )	1.0	1.0	1.0	0.6	1.0	0.8
Max force roof-beam (kN)	31	37	46	59	51	91
Relative ( )	1.9	0.4	2.0	0.4	2.0	0.5
Max force beam-column (kN)	57	33	89	53	106	82
Relative ( )	1.0	0.5	1.0	0.5	1.0	0.7
Total column shear (kN)	1466	1297	1990	1610	2174	2052
Relative ( )	1.0	0.9	1.0	0.8	1.0	0.9
Mean column shear (kN)	73	65	100	80	109	103
Relative ( )	1.0	0.9	1.0	0.8	1.0	0.9
Max column shear (kN)	79	85	102	103	117	117
Relative ( )	1.07	1.32	1.03	1.28	1.07	1.14

Table 4b

Vertical panels - 1-bay - isostatic connections - deformable diaphragm

	ISO3/1DEF	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
		y direction	**x direction**	**y direction**	**x direction**	**y direction**	**x direction**
a1	Maximum top drift (mm)	97	105	184	204	237	249
a2	Ratio (%)	1.3	1.4	2.4	2.7	3.2	3.3
b1	Differential top drift (mm)	0	23	0	42	0	46
b2	Ratio (%)	0.0	0.3	0.0	0.6	0.0	0.6
c1	Maximum top drift (mm)	97	105	184	204	237	249
c2	Relative ( )	1.01	1.05	1.01	1.07	1.00	1.08
d1	Max connection slide (mm)	97	82	184	162	237	203
d2	Relative ( )	1.01	0.82	1.01	0.85	1.00	0.88
f1	Max force roof-beam (kN)	25	169	30	256	33	265
f2	Relative ( )	0.36	2.31	0.33	2.72	0.34	2.73
g1	Max force beam-column (kN)	75	80	96	98	98	100
g2	Relative ( )	1.01	1.04	1.00	1.01	1.00	1.01
k1	Total column shear (kN)	1430	1434	1863	1886	1907	1933
k2	Relative ( )	1.01	0.99	1.00	1.00	1.00	1.00
l1	Mean column shear (kN)	71	72	93	94	95	97
l2	Relative ( )	1.00	0.99	1.00	1.00	0.99	1.00
m1	Max column shear (kN)	75	80	96	98	98	100
m2	Relative ( )	1.06	1.11	1.03	1.04	1.03	1.03

Table 5a

Horizontal panels - 1-bay - isostatic connections - deformable diaphragm

	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
	y direction	**x direction**	**y direction**	**x direction**	**y direction**	**x direction**
Maximum top drift (mm)	94	98	183	191	235	236
Ratio (%)	1.3	1.3	2.4	2.5	3.1	3.1
Differential top drift (mm)	0	5	0	10	0	12
Ratio (%)	0.0	0.7	0.0	1.3	0.0	1.5
Maximum top drift (mm)	94	98	183	191	235	236
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max connection slide (mm)	33	33	61	57	76	71
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max force roof-beam (kN)	18	125	27	225	30	241
Relative ( )	1.1	1.3	1.2	1.5	1.2	1.3
Max force beam-column (kN)	57	62	89	106	106	112
Relative ( )	1.0	1.0	1.0	1.1	1.0	1.0
Total column shear (kN)	1471	1502	2001	2015	2177	2231
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Mean column shear (kN)	74	75	100	101	109	112
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max column shear (kN)	79	82	103	104	117	118
Relative ( )	1.07	1.09	1.03	1.03	1.07	1.06

Table 5b

Vertical panels - 1-bay - isostatic connections - rigid diaphragm

	ISO3/1RIG	**PGA = 0.18 g**		**PGA = 0.36 g**		**PGA = 0.60 g**	
		y direction	**x direction**	**y direction**	**x direction**	**y direction**	**x direction**
a1	Maximum top drift (mm)	96	100	183	191	236	230
a2	Ratio (%)	1.3	1.3	2.4	2.5	3.1	3.1
b1	Differential top drift (mm)	0	8	0	14	0	16
b2	Ratio (%)	0.0	0.1	0.0	0.2	0.0	0.2
c1	Maximum top drift (mm)	96	100	183	191	236	230
c2	Relative ( )	1.00	1.00	1.00	1.00	1.00	1.00
d1	Max connection slide (mm)	96	100	183	191	236	230
d2	Relative ( )	1.00	1.00	1.00	1.00	1.00	1.00
f1	Max force roof-beam (kN)	27	135	81	192	32	205
f2	Relative ( )	0.38	1.85	0.33	2.04	0.33	2.12
g1	Max force beam-column (kN)	74	77	96	97	98	99
g2	Relative ( )	1.00	1.00	1.00	1.00	1.00	1.00
k1	Total column shear (kN)	1419	1453	1865	1886	1910	1939
k2	Relative ( )	1.00	1.00	1.00	1.00	1.00	1.00
l1	Mean column shear (kN)	71	73	93	94	96	97
l2	Relative ( )	1.00	1.00	1.00	1.00	1.00	1.00
m1	Max column shear (kN)	74	77	96	97	98	99
m2	Relative ( )	1.04	1.05	1.03	1.03	1.02	1.02

Table 6a

Horizontal panels - 1-bay - isostatic connections - rigid diaphragm

	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
	y direction	**x direction**	**y direction**	**x direction**	**y direction**	**x direction**
Maximum top drift (mm)	95	94	183	185	236	231
Ratio (%)	1.3	1.3	2.4	2.5	3.1	3.1
Differential top drift (mm)	0	2	0	3	0	4
Ratio (%)	0.0	0.2	0.0	0.4	0.0	0.5
Maximum top drift (mm)	95	94	183	185	236	231
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max connection slide (mm)	33	32	61	59	76	72
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max force roof-beam (kN)	16	97	23	155	25	187
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max force beam-column (kN)	58	64	89	101	105	115
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Total column shear (kN)	1473	1515	2003	2026	2178	2271
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Mean column shear (kN)	74	76	100	101	109	114
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max column shear (kN)	79	79	103	104	117	119
Relative ( )	1.07	1.04	1.03	1.02	1.07	1.05

Table 6b

Vertical panels - 2-bays - isostatic connections - null-diaphragm

	ISO3/2NUL	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
		y direction	x direction	y direction	x direction	y direction	x direction
a1	Maximum top drift (mm)	151	156	261	218	372	316
a2	Ratio (%)	2.0	2.1	3.5	2.9	5.0	4.2
b1	Differential top drift (mm)	110	147	214	194	387	347
b2	Ratio (%)	1.5	2.0	2.8	2.6	5.2	4.6
c1	Maximum top drift (mm)	151	156	261	218	372	316
c2	Relative ( )	1.54	1.59	1.40	1.17	1.64	1.38
d1	Max connection slide (mm)	55	32	73	90	145	145
d2	Relative ( )	0.56	0.33	0.39	0.49	0.64	0.63
f1	Max force roof-beam (kN)	32	65	42	90	50	141
f2	Relative ( )	0.43	0.87	0.44	0.93	0.51	1.42
g1	Max force beam-column (kN)	104	106	111	112	113	113
g2	Relative ( )	1.39	1.41	1.16	1.15	1.15	1.14
k1	Total column shear (kN)	1820	1562	2306	2468	2921	3003
k2	Relative ( )	0.78	0.67	0.78	0.82	0.96	0.98
l1	Mean column shear (kN)	59	50	74	80	94	97
l2	Relative ( )	0.79	0.67	0.77	0.82	0.96	0.98
m1	Max column shear (kN)	104	106	111	112	113	113
m2	Relative ( )	1.76	2.12	1.50	1.40	1.20	1.16

Table 7a

Horizontal panels - 2-bays - isostatic connections - null-diaphragm

	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
	y direction	x direction	y direction	x direction	y direction	x direction
Maximum top drift (mm)	122	122	242	206	339	305
Ratio (%)	1.6	1.6	3.2	2.7	4.5	4.1
Differential top drift (mm)	0	64	0	136	0	226
Ratio (%)	0.0	8.6	0.0	18.2	0.0	30.2
Maximum top drift (mm)	122	122	242	206	339	305
Relative ( )	1.3	1.3	1.3	1.1	1.4	1.3
Max connection slide (mm)	23	19	32	34	63	52
Relative ( )	0.7	0.6	0.5	0.6	0.8	0.7
Max force roof-beam (kN)	25	40	32	68	47	104
Relative ( )	0.8	0.4	0.8	0.4	0.8	0.6
Max force beam-column (kN)	46	37	65	61	91	94
Relative ( )	0.7	0.6	0.6	0.6	0.7	0.8
Total column shear (kN)	2000	2123	2612	2554	2944	3009
Relative ( )	0.8	0.9	0.9	0.8	0.9	0.9
Mean column shear (kN)	65	68	84	82	95	97
Relative ( )	0.8	0.9	0.9	0.8	0.9	0.9
Max column shear (kN)	99	98	108	106	121	115
Relative ( )	1.53	1.44	1.28	1.29	1.27	1.19

Table 7b

Vertical panels - 2-bays - isostatic connections - deformable diaphragm

	ISO3/2DEF	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
		y direction	x direction	y direction	x direction	y direction	x direction
a1	Maximum top drift (mm)	99	100	188	188	230	229
a2	Ratio (%)	1.3	1.3	2.5	2.5	3.1	3.1
b1	Differential top drift (mm)	1	4	2	6	2	7
b2	Ratio (%)	0.0	0.1	0.0	0.1	0.0	0.1
c1	Maximum top drift (mm)	99	100	188	188	230	229
c2	Relative ( )	1.01	1.02	1.01	1.01	1.01	1.00
d1	Max connection slide (mm)	100	100	186	182	228	222
d2	Relative ( )	1.02	1.03	1.0	0.98	1.00	0.97
f1	Max force roof-beam (kN)	107	185	135	250	149	254
f2	Relative ( )	1.42	2.46	1.40	2.58	1.52	2.56
g1	Max force beam-column (kN)	87	87	107	108	110	111
g2	Relative ( )	1.16	1.16	1.11	1.11	1.12	1.12
k1	Total column shear (kN)	2319	2326	2979	3008	3059	3086
k2	Relative ( )	0.99	1.00	1.00	1.00	1.00	1.00
l1	Mean column shear (kN)	75	75	96	97	99	100
l2	Relative ( )	1.00	1.00	1.00	1.00	1.01	1.01
m1	Max column shear (kN)	87	87	107	108	110	111
m2	Relative ( )	1.16	1.16	1.11	1.11	1.11	1.11

Table 8a

Horizontal panels - 2-bays - isostatic connections - deformable diaphragm

	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
	y direction	**x direction**	**y direction**	**x direction**	**y direction**	**x direction**
Maximum top drift (mm)	99	98	182	182	243	241
Ratio (%)	1.3	1.3	2.4	2.4	3.2	3.2
Differential top drift (mm)	0	1	0	2	0	2
Ratio (%)	0.0	0.1	0.0	0.2	0.0	0.3
Maximum top drift (mm)	99	98	182	182	243	241
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max connection slide (mm)	36	31	62	55	79	71
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max force roof-beam (kN)	71	130	106	203	141	253
Relative ( )	2.2	1.3	2.7	1.3	2.4	1.4
Max force beam-column (kN)	72	65	112	100	116	111
Relative ( )	1.1	1.0	1.1	1.0	0.9	0.9
Total column shear (kN)	2411	2428	3071	3050	3337	3306
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Mean column shear (kN)	78	78	99	98	108	107
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max column shear (kN)	87	87	107	107	113	114
Relative ( )	1.12	1.11	1.08	1.08	1.05	1.07

Table 8b

Vertical panels - 2-bays - isostatic connections - rigid diaphragm

	ISO3/2RIG	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
		y direction	x direction	y direction	x direction	y direction	x direction
a1	Maximum top drift (mm)	98	98	186	186	227	229
a2	Ratio (%)	1.3	1.3	2.5	2.5	3.0	3.1
b1	Differential top drift (mm)	0	1	0	1	0	1
b2	Ratio (%)	0.0	0.0	0.0	0.0	0.0	0.0
c1	Maximum top drift (mm)	98	98	186	186	227	229
c2	Relative ( )	1.00	1.00	1.00	1.00	1.00	1.00
d1	Max connection slide (mm)	98	97	186	185	227	228
d2	Relative ( )	1.00	0.99	1.00	0.99	1.00	1.00
f1	Max force roof-beam (kN)	101	102	133	142	130	142
f2	Relative ( )	1.34	1.35	1.89	1.46	1.33	1.43
g1	Max force beam-column (kN)	85	84	106	106	107	108
g2	Relative ( )	1.13	1.12	1.10	1.09	1.09	1.09
k1	Total column shear (kN)	2337	2316	2973	2996	3045	3074
k2	Relative ( )	1.00	1.00	1.00	1.00	1.00	1.00
l1	Mean column shear (kN)	75	75	96	97	98	99
l2	Relative ( )	1.00	1.00	1.00	1.00	1.00	1.00
m1	Max column shear (kN)	85	84	106	106	107	108
m2	Relative ( )	1.13	1.12	1.10	1.09	1.09	1.09

Table 9a

Horizontal panels - 2-bays - isostatic connections - rigid diaphragm

	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
	y direction	**x direction**	**y direction**	**x direction**	**y direction**	**x direction**
Maximum top drift (mm)	97	97	181	181	239	239
Ratio (%)	1.3	1.3	2.4	2.4	3.2	3.2
Differential top drift (mm)	0	0	0	0	0	0
Ratio (%)	0.0	0.0	0.0	0.0	0.0	0.0
Maximum top drift (mm)	97	97	181	181	239	239
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max connection slide (mm)	35	31	61	56	77	71
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max force roof-beam (kN)	32	100	39	153	60	187
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max force beam-column (kN)	68	66	104	101	125	122
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Total column shear (kN)	2431	2433	3068	3069	3343	3335
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Mean column shear (kN)	78	78	99	99	108	108
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max column shear (kN)	86	84	106	104	113	113
Relative ( )	1.09	1.07	1.07	1.06	1.05	1.05

Table 9b

Vertical panel - 3-bays - isostatic connections - null diaphragm

	ISO3/3NUL	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
		y direction	x direction	y direction	x direction	y direction	x direction
a1	Maximum top drift (mm)	124	150	220	211	327	310
a2	Ratio (%)	1.7	2.0	2.9	2.8	4.4	4.1
b1	Differential top drift (mm)	94	149	201	211	321	276
b2	Ratio (%)	1.3	2.0	2.7	2.8	4.3	3.7
c1	Maximum top drift (mm)	124	150	220	211	327	310
c2	Relative ( )	1.19	1.44	1.12	1.09	1.19	1.30
d1	Max connection slide (mm)	42	43	130	96	220	100
d2	Relative ( )	0.40	0.41	0.66	0.49	0.80	0.42
f1	Max force roof-beam (kN)	31	88	47	97	58	129
f2	Relative ( )	0.40	1.21	0.49	1.05	0.59	1.32
g1	Max force beam-column (kN)	99	102	107	108	112	113
g2	Relative ( )	1.29	1.40	1.11	1.17	1.14	1.15
k1	Total column shear (kN)	2705	2634	3853	3660	3663	3535
k2	Relative ( )	0.84	0.86	0.96	0.94	0.89	0.85
l1	Mean column shear (kN)	64	63	92	87	87	84
l2	Relative ( )	0.83	0.86	0.96	0.95	0.89	0.86
m1	Max column shear (kN)	99	102	107	108	112	113
m2	Relative ( )	1.55	1.62	1.167	1.24	1.29	1.35

Table 10a

Horizontal panel - 3-bays - isostatic connections - null diaphragm

	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
	y direction	**x direction**	**y direction**	**x direction**	**y direction**	**x direction**
Maximum top drift (mm)	110	115	215	221	287	271
Ratio (%)	1.5	1.5	2.9	2.9	3.8	3.6
Differential top drift (mm)	87	64	204	158	192	214
Ratio (%)	11.6	8.5	27.1	21.1	25.6	28.5
Maximum top drift (mm)	110	115	215	221	287	271
Relative ( )	1.1	1.2	1.2	1.2	1.1	1.1
Max connection slide (mm)	21	13	42	30	71	54
Relative ( )	0.6	0.4	0.7	0.5	0.9	0.7
Max force roof-beam (kN)	24	41	38	81	47	122
Relative ( )	0.4	0.3	0.3	0.4	0.2	0.5
Max force beam-column (kN)	76	45	120	71	148	111
Relative ( )	1.1	0.5	1.1	0.5	1.1	0.7
Total column shear (kN)	2637	2570	3670	3073	3812	3188
Relative ( )	0.9	0.8	1.0	0.8	0.9	0.8
Mean column shear (kN)	63	61	87	73	91	76
Relative ( )	0.9	0.8	1.0	0.8	0.9	0.8
Max column shear (kN)	93	95	109	108	110	112
Relative ( )	1.48	1.55	1.24	1.48	1.22	1.48

Table 10b

Vertical panel - 3-bays - isostatic connections - deformable diaphragm

	ISO3/3DEF	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
		y direction	x direction	y direction	x direction	y direction	x direction
a1	Maximum top drift (mm)	105	110	190	198	236	248
a2	Ratio (%)	1.4	1.5	2.5	2.6	3.1	3.3
b1	Differential top drift (mm)	6	25	7	45	10	48
b2	Ratio (%)	0.1	0.3	0.1	0.6	0.1	0.6
c1	Maximum top drift (mm)	105	110	190	198	236	248
c2	Relative ( )	1.01	1.06	0.97	1.02	0.86	1.04
d1	Max connection slide (mm)	100	85	183	153	226	200
d2	Relative ( )	0.96	0.82	0.93	0.79	0.82	0.84
f1	Max force roof-beam (kN)	184	197	190	196	282	291
f2	Relative ( )	2.38	2.70	1.78	2.13	2.87	2.97
g1	Max force beam-column (kN)	91	94	110	109	111	111
g2	Relative ( )	1.18	1.29	1.15	1.18	1.13	1.13
k1	Total column shear (kN)	3247	3201	4126	4073	4221	4183
k2	Relative ( )	1.00	1.05	1.03	1.02	1.09	1.01
l1	Mean column shear (kN)	77	76	98	97	100	100
l2	Relative ( )	1.00	1.04	1.02	1.05	1.02	1.02
m1	Max column shear (kN)	91	94	110	109	111	111
m2	Relative (%)	1.18	1.24	1.12	1.12	1.11	1.11

Table 11a

Horizontal panel - 3-bays - isostatic connections - deformable diaphragm

	PGA = 0.18 g		**PGA = 0.36 g**		**PGA = 0.60 g**	
	y direction	**x direction**	**y direction**	**x direction**	**y direction**	**x direction**
Maximum top drift (mm)	103	104	180	193	259	248
Ratio (%)	1.4	1.4	2.4	2.6	3.5	3.3
Differential top drift (mm)	3	8	6	14	8	14
Ratio (%)	0.4	1.0	0.7	1.9	1.1	1.9
Maximum top drift (mm)	103	104	180	193	259	248
Relative ( )	1.0	1.1	1.0	1.1	1.0	1.0
Max connection slide (mm)	35	31	60	53	82	73
Relative ( )	1.0	0.9	1.0	0.9	1.0	1.0
Max force roof-beam (kN)	97	167	147	291	248	304
Relative ( )	1.6	1.3	1.2	1.4	1.2	1.3
Max force beam-column (kN)	72	89	108	146	135	159
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Total column shear (kN)	3077	3170	3840	3990	4192	4047
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Mean column shear (kN)	73	75	91	95	100	96
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max column shear (kN)	89	90	106	107	110	120
Relative ( )	1.21	1.19	1.16	1.12	1.10	1.24

Table 11 b

Vertical panel - 3-bays - isostatic connections - rigid diaphragm

	ISO3/3RIG	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
		y direction	x direction	y direction	x direction	y direction	x direction
a1	Maximum top drift (mm)	104	104	196	194	275	238
a2	Ratio (%)	1.4	1.4	2.6	2.6	3.7	3.2
b1	Differential top drift (mm)	3	9	14	15	20	17
b2	Ratio (%)	0.0	0.1	0.2	0.2	0.3	0.2
c1	Maximum top drift (mm)	104	104	196	194	275	238
c2	Relative ( )	1.00	1.00	1.00	1.00	1.00	1.00
d1	Max connection slide (mm)	101	95	182	179	255	211
d2	Relative ( )	0.97	0.91	0.93	0.92	0.93	0.86
f1	Max force roof-beam (kN)	66	173	226	237	325	265
f2	Relative ( )	0.86	2.38	2.35	2.58	3.32	2.71
g1	Max force beam-column (kN)	89	89	107	107	110	109
g2	Relative ( )	1.16	1.22	1.11	1.16	1.12	1.11
k1	Total column shear (kN)	3239	3059	4012	3881	4111	4136
k2	Relative ( )	1.00	1.00	1.00	1.00	1.00	1.00
l1	Mean column shear (kN)	77	73	96	92	98	98
l2	Relative ( )	1.00	1.00	1.00	1.00	1.00	1.00
m1	Max column shear (kN)	89	89	107	107	110	109
m2	Relative ( )	1.16	1.32	1.11	1.16	1.12	1.11

Table 12a

Horizontal panel - 3-bays - isostatic connections - rigid diaphragm

	PGA = 0.18 g		PGA = 0.36 g		PGA = 0.60 g	
	y direction	x direction	y direction	x direction	y direction	x direction
Maximum top drift (mm)	102	99	179	183	256	240
Ratio (%)	1.4	1.3	2.4	2.4	3.4	3.2
Differential top drift (mm)	0	2	1	4	1	4
Ratio (%)	0.0	0.3	0.1	0.5	0.2	0.5
Maximum top drift (mm)	102	99	179	183	256	240
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max connection slide (mm)	36	34	60	59	82	75
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max force roof-beam (kN)	59	130	120	202	201	231
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max force beam-column (kN)	72	91	111	140	139	159
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Total column shear (kN)	3082	3223	3845	4025	4185	4194
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Mean column shear (kN)	73	77	92	96	100	100
Relative ( )	1.0	1.0	1.0	1.0	1.0	1.0
Max column shear (kN)	88	86	106	106	112	113
Relative ( )	1.20	1.13	1.16	1.10	1.12	1.13

Table 12b

Comments

ONE STOREY BUILDING WITH VERTICAL PANELS

At service (0,18g) limit conditions, maximum drifts from 1,3% to 2,1% have been evaluated, the smaller for rigid diaphragm (that is from 96 mm to 156 mm).

At no-collapse (0,36g) limit conditions, drifts from 2,4% to 3,5% have been evaluated, the smaller for rigid diaphragm (that is from 183 mm to 261 mm).

The maximum panel-to-structure connection slides are of the same magnitude of the drifts above indicated (for a "cantilever" arrangement of the panels with sliding upper connections).

At service (0,18g) limit conditions, maximum roof-to-beam forces from 31 kN to 88 kN have been evaluated for null diaphragm, from 25 kN to 197 kN for deformable and rigid diaphragms.

At no-collapse (0,36g) limit conditions, maximum roof-to-beam forces from 70 kN to 97 kN have been evaluated for null diaphragm, from 30 kN to 256 kN for deformable and rigid diaphragms .

At service (0,18g) limit conditions, maximum beam-to-column forces from 74 kN to 106 kN have been evaluated for all diaphragm types.

At no-collapse (0,36g) limit condition, maximum beam-to-column forces from 96 kN to 112 kN have been evaluated for all diaphragm types.

ONE STOREY BUILDINGS WITH HORIZONTAL PANELS

At service (0,18g) limit conditions, maximum drifts from 1,3% to 1,6% have been evaluated, the smaller for rigid diaphragm (that is from 94 mm to 122 mm).

At no-collapse (0,36g) limit conditions, drifts from 2,4% to 3,2% have been evaluated, the smaller for rigid diaphragm (that is from 182 mm to 242 mm).

At service (0,18g) limit conditions, the maximum panel-to-structure connection slides from 19 to 36 mm have been evaluated, the smaller for null diaphragm.

At no-collapse (0,36g) limit conditions, the maximum panel-to-structure connection slides from 30 to 62 have been evaluated, the smaller for null diaphragm.

At service (0,18g) limit conditions, maximum roof-to-beam forces from 24 kN to 40 kN have been evaluated for null diaphragm, from 16 kN to 167 kN for deformable and rigid diaphragms.

At no-collapse (0,36g) limit conditions, maximum roof-to-beam forces from 32kN to 81 kN have been evaluated for null diaphragm, from 23 kN to 291 kN for deformable and rigid diaphragms.

At service (0,18g) limit conditions, maximum beam-to-column forces from 33 kN to 91 kN have been evaluated for all diaphragm types.

At no-collapse (0,36g) limit condition, maximum beam-to-column forces from 53 kN to 146 kN have been evaluated for all diaphragm types.

GENERAL COMMENTS

The isostatic solution of panel-to-structure connections saves all the current criteria of the present design practice, except for the need of sliding connections with very wide free slide capacity: from ±50 mm to ±101 mm at service (0,18g) limit conditions, from ±73 mm to ±191 mm at no-collapse (0,36g) limit conditions

Maximum forces in roof-to-beam connections are indeed very large in some cases. Maximum forces are particularly large in the transverse direction of the structure with semi-rigid diaphragm. However, these forces were obtained with the numerical model where the unlimited strength and stiffness of the connections were taken into account.

In reality, when these connections are damaged the forces should be redistributed to less loaded connections, and the final forces would be in average smaller (please note that the maximum forces occur only in limited number of connections).

1-bay – integrated connections - null diaphragm – three connections per panel.

	INT3/1NUL	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	9	25	97	45	85	148
a2	ratio [%]	0.1	0.3	1.3	0.6	1.1	2.0
b1	Differential top drift [mm]	0	0	0	45	85	147
b2	ratio [%]	0.0	0.0	0.0	0.6	1.1	2.0
c1	Maximum top drift [mm]	9	25	97	45	85	148
c2	Relative ( )	0.09	0.14	0.41	0.45	0.45	0.64
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
f1	Max horizontal force roof-beam [kN]	82	95	91	98	119	146
f2	Relative ( )	1.15	1.02	0.95	1.34	1.27	1.50
g1	Max horizontal force beam-column [kN]	77	102	122	131	177	246
g2	Relative ( )	1.09	1.09	1.27	1.79	1.89	2.54
h1	Max force panel-beam [kN]	318	406	446	183	318	505
h2	Relative ( )	4.48	4.36	4.65	2.50	3.38	5.21
j1	Total base shear [kN]	4231	6660	8367	1460	2772	4689
j2	Relative ( )	2.98	3.57	4.38	1.00	1.47	2.42
k1	Total column shear [kN]	450	718	1218	541	699	859
k2	Relative ( )	0.11	0.11	0.15	0.37	0.25	0.18
l1	Mean column shear [kN]	23	36	61	27	35	43
l2	Relative ( )	0.32	0.39	0.63	0.37	0.37	0.44
m1	Max column shear [kN]	24	37	61	44	61	74
m2	Relative ( )	1.07	1.03	1.01	1.62	1.73	1.73

Table 13a

1-bay – integrated connections - null diaphragm – four connections per panel.

	INT3/1NUL	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	4	10	21	45	87	148
a2	ratio [%]	0.1	0.1	0.3	0.6	1.2	2.0
b1	Differential top drift [mm]	0	0	0	45	87	148
b2	ratio [%]	0.0	0.0	0.0	0.6	1.2	2.0
c1	Maximum top drift [mm]	4	10	21	45	87	148
c2	Relative ( )	0.04	0.05	0.09	0.45	0.45	0.64
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
f1	Max horizontal force roof-beam [kN]	78	98	129	93	111	135
f2	Relative ( )	1.10	1.06	1.35	1.27	1.18	1.40
g1	Max horizontal force beam-column [kN]	76	106	150	129	199	286
g2	Relative ( )	1.08	1.14	1.56	1.77	2.12	2.95
h1	Max force panel-beam [kN]	188	286	448	153	227	324
h2	Relative ( )	2.65	3.08	4.67	2.09	2.41	3.34
j1	Total base shear [kN]	3907	7250	11480	1305	2603	4209
j2	Relative ( )	2.75	3.89	6.01	0.90	1.38	2.17
k1	Total column shear [kN]	219	485	685	542	701	865
k2	Relative ( )	0.06	0.07	0.06	0.42	0.27	0.21
l1	Mean column shear [kN]	11	24	34	27	35	43
l2	Relative ( )	0.15	0.26	0.36	0.37	0.37	0.45
m1	Max column shear [kN]	13	26	35	44	61	74
m2	Relative ( )	1.18	1.09	1.03	1.62	1.74	1.71

Table 13b

1-bay – integrated connections - deformable diaphragm – three connections per panel.

	INT3/1DEF	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	8	13	77	38	83	147
a2	ratio [%]	0.1	0.2	1.0	0.5	1.1	2.0
b1	Differential top drift [mm]	0	0	0	32	68	119
b2	ratio [%]	0.0	0.0	0.0	0.4	0.9	1.6
c1	Maximum top drift [mm]	8	13	77	38	83	147
c2	Relative ( )	0.08	0.07	0.33	0.38	0.43	0.64
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
f1	Max horizontal force roof-beam [kN]	124	146	154	656	1443	2483
f2	Relative ( )	1.74	1.57	1.61	8.99	15.36	25.60
g1	Max horizontal force beam-column [kN]	81	100	157	424	942	1666
g2	Relative ( )	1.14	1.07	1.63	5.81	10.03	17.18
h1	Max force panel-beam [kN]	285	372	408	304	601	972
h2	Relative ( )	4.01	4.00	4.25	4.16	6.39	10.02
j1	Total base shear [kN]	3796	6362	8180	1785	3891	6210
j2	Relative ( )	2.68	3.41	4.28	1.23	2.06	3.20
k1	Total column shear [kN]	404	516	995	541	790	1047
k2	Relative ( )	0.11	0.08	0.12	0.30	0.20	0.17
l1	Mean column shear [kN]	20	26	50	27	40	52
l2	Relative ( )	0.28	0.28	0.52	0.37	0.42	0.54
m1	Max column shear [kN]	23	29	51	34	47	66
m2	Relative ( )	1.16	1.13	1.03	1.25	1.19	1.26

Table 14a

1-bay – integrated connections - deformable diaphragm – four connections per panel.

	INT3/1DEF	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	4	8	14	36	77	139
a2	ratio [%]	0.0	0.1	0.2	0.5	1.0	1.9
b1	Differential top drift [mm]	0	0	0	33	71	126
b2	ratio [%]	0.0	0.0	0.0	0.4	0.9	1.7
c1	Maximum top drift [mm]	4	8	14	36	77	139
c2	Relative ( )	0.04	0.04	0.06	0.36	0.40	0.61
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
f1	Max horizontal force roof-beam [kN]	117	154	196	696	1481	2569
f2	Relative ( )	1.64	1.66	2.05	9.53	15.76	26.48
g1	Max horizontal force beam-column [kN]	95	128	181	425	938	1699
g2	Relative ( )	1.34	1.38	1.89	5.83	9.98	17.51
h1	Max force panel-beam [kN]	178	256	340	222	366	582
h2	Relative ( )	2.51	2.75	3.54	3.04	3.89	6.00
j1	Total base shear [kN]	3669	6491	10008	2003	3517	6216
j2	Relative ( )	2.59	3.48	5.24	1.38	1.86	3.21
k1	Total column shear [kN]	219	387	580	473	687	957
k2	Relative ( )	0.06	0.06	0.06	0.24	0.20	0.15
l1	Mean column shear [kN]	11	19	29	24	34	48
l2	Relative ( )	0.15	0.21	0.30	0.32	0.37	0.49
m1	Max column shear [kN]	14	26	33	35	45	63
m2	Relative ( )	1.30	1.32	1.14	1.46	1.30	1.32

Table 14b

1-bay – integrated connections - rigid diaphragm – three connections per panel.

	INT3/1RIG	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	9	17	86	31	60	124
a2	ratio [%]	0.1	0.2	1.1	0.4	0.8	1.6
b1	Differential top drift [mm]	0	0	0	21	33	45
b2	ratio [%]	0.0	0.0	0.0	0.3	0.4	0.6
c1	Maximum top drift [mm]	9	17	86	31	60	124
c2	Relative ( )	0.10	0.09	0.36	0.31	0.32	0.54
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
e1	Max force roof-roof [kN]	31	36	39	211	399	522
e2	Relative ( )	0.44	0.38	0.40	2.89	4.24	5.38
f1	Max horizontal force roof-beam [kN]	118	129	135	671	1201	1510
f2	Relative ( )	1.66	1.39	1.41	9.19	12.78	15.57
g1	Max horizontal force beam-column [kN]	91	105	150	818	1351	1923
g2	Relative ( )	1.29	1.13	1.56	11.21	14.37	19.83
h1	Max force panel-beam [kN]	334	376	418	524	946	1094
h2	Relative ( )	4.71	4.05	4.36	7.18	10.06	11.28
j1	Total base shear [kN]	4498	6515	8298	2944	4969	6146
j2	Relative ( )	3.17	3.49	4.34	2.03	2.63	3.17
k1	Total column shear [kN]	402	555	1123	577	746	1450
k2	Relative ( )	0.09	0.09	0.14	0.20	0.15	0.24
l1	Mean column shear [kN]	20	28	56	29	37	73
l2	Relative ( )	0.28	0.30	0.58	0.40	0.40	0.75
m1	Max column shear [kN]	25	31	56	36	48	82
m2	Relative ( )	1.23	1.11	1.00	1.24	1.28	1.13

Table 15a

1-bay – integrated connections - rigid diaphragm – four connections per panel.

	INT3/1RIG	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	5	10	17	35	65	98
a2	ratio [%]	0.1	0.1	0.2	0.5	0.9	1.3
b1	Differential top drift [mm]	0	0	0	27	44	56
b2	ratio [%]	0.0	0.0	0.0	0.4	0.6	0.7
c1	Maximum top drift [mm]	5	10	17	35	65	98
c2	Relative ( )	0.05	0.05	0.07	0.35	0.34	0.42
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
e1	Max force roof-roof [kN]	30	41	55	303	497	711
e2	Relative ( )	0.42	0.44	0.57	4.15	5.29	7.33
f1	Max horizontal force roof-beam [kN]	115	141	174	922	1468	2081
f2	Relative ( )	1.62	1.52	1.81	12.63	15.62	21.45
g1	Max horizontal force beam-column [kN]	102	135	173	1045	1795	2241
g2	Relative ( )	1.44	1.45	1.80	14.31	19.09	23.10
h1	Max force panel-beam [kN]	212	287	388	435	675	971
h2	Relative ( )	2.98	3.09	4.04	5.96	7.18	10.01
j1	Total base shear [kN]	4469	7605	11516	3809	6350	8633
j2	Relative ( )	3.15	4.08	6.03	2.62	3.37	4.45
k1	Total column shear [kN]	260	454	568	476	645	761
k2	Relative ( )	0.06	0.06	0.05	0.12	0.10	0.09
l1	Mean column shear [kN]	13	23	28	24	32	38
l2	Relative ( )	0.18	0.24	0.30	0.33	0.34	0.39
m1	Max column shear [kN]	15	29	34	32	42	51
m2	Relative ( )	1.19	1.27	1.19	1.36	1.29	1.34

Table 15b

2-bay – integrated connections - null diaphragm – three connections per panel.

	INT3/2NUL	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	48	99	180	49	87	144
a2	ratio [%]	0.6	1.3	2.4	0.7	1.2	1.9
b1	Differential top drift [mm]	49	99	194	50	89	147
b2	ratio [%]	0.6	1.3	2.6	0.7	1.2	2.0
c1	Maximum top drift [mm]	48	99	180	49	87	144
c2	Relative ( )	0.49	0.53	0.79	0.50	0.47	0.63
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
f1	Max horizontal force roof-beam [kN]	94	101	114	140	199	266
f2	Relative ( )	1.25	1.05	1.16	1.87	2.05	2.69
g1	Max horizontal force beam-column [kN]	110	216	323	177	243	325
g2	Relative ( )	1.46	2.25	3.29	2.36	2.51	3.28
h1	Max force panel-beam [kN]	218	387	607	159	272	440
h2	Relative ( )	2.91	4.03	6.19	2.12	2.81	4.45
j1	Total base shear [kN]	2881	5916	10304	2118	3971	6474
j2	Relative ( )	1.23	1.99	3.38	0.91	1.33	2.11
k1	Total column shear [kN]	603	842	1264	910	1153	1480
k2	Relative ( )	0.21	0.14	0.12	0.43	0.29	0.23
l1	Mean column shear [kN]	19	27	41	29	37	48
l2	Relative ( )	0.26	0.28	0.42	0.39	0.38	0.48
m1	Max column shear [kN]	54	73	89	49	65	87
m2	Relative ( )	2.78	2.70	2.19	1.68	1.76	1.82

Table 16a

2-bay – integrated connections - null diaphragm – four connections per panel.

	INT3/2NUL	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	51	105	164	50	88	145
a2	ratio [%]	0.7	1.4	2.2	0.7	1.2	1.9
b1	Differential top drift [mm]	51	106	171	50	89	145
b2	ratio [%]	0.7	1.4	2.3	0.7	1.2	1.9
c1	Maximum top drift [mm]	51	105	164	50	88	145
c2	Relative ( )	0.52	0.56	0.72	0.51	0.47	0.63
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
f1	Max horizontal force roof-beam [kN]	93	97	106	133	173	233
f2	Relative ( )	1.24	1.01	1.08	1.78	1.78	2.35
g1	Max horizontal force beam-column [kN]	132	254	388	173	248	352
g2	Relative ( )	1.76	2.65	3.96	2.30	2.56	3.55
h1	Max force panel-beam [kN]	145	224	324	137	198	283
h2	Relative ( )	1.93	2.33	3.30	1.83	2.04	2.86
j1	Total base shear [kN]	2803	5381	9117	1891	3648	5972
j2	Relative ( )	1.20	1.81	2.99	0.82	1.22	1.94
k1	Total column shear [kN]	608	857	1173	915	1177	1494
k2	Relative ( )	0.22	0.16	0.13	0.48	0.32	0.25
l1	Mean column shear [kN]	20	28	38	30	38	48
l2	Relative ( )	0.26	0.29	0.39	0.39	0.39	0.49
m1	Max column shear [kN]	55	76	88	49	65	86
m2	Relative ( )	2.82	2.73	2.33	1.66	1.71	1.78

Table 16b

2-bay – integrated connections - deformable diaphragm – three connections per panel.

	INT3/2DEF	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	26	55	85	41	88	153
a2	ratio [%]	0.3	0.7	1.1	0.5	1.2	2.0
b1	Differential top drift [mm]	19	42	56	36	77	130
b2	ratio [%]	0.3	0.6	0.7	0.5	1.0	1.7
c1	Maximum top drift [mm]	26	55	85	41	88	153
c2	Relative ( )	0.27	0.30	0.38	0.42	0.47	0.67
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
f1	Max horizontal force roof-beam [kN]	537	996	1314	762	1469	2392
f2	Relative ( )	7.16	10.37	13.41	10.16	15.15	24.16
g1	Max horizontal force beam-column [kN]	246	430	556	416	928	1599
g2	Relative ( )	3.27	4.48	5.67	5.54	9.57	16.15
h1	Max force panel-beam [kN]	297	557	680	257	491	761
h2	Relative ( )	3.97	5.80	6.94	3.43	5.07	7.69
j1	Total base shear [kN]	4186	8195	11384	2711	5660	9794
j2	Relative ( )	1.79	2.76	3.74	1.17	1.89	3.19
k1	Total column shear [kN]	766	1094	1339	855	1233	1629
k2	Relative ( )	0.18	0.13	0.12	0.32	0.22	0.17
l1	Mean column shear [kN]	25	35	43	28	40	53
l2	Relative ( )	0.33	0.37	0.44	0.37	0.41	0.53
m1	Max column shear [kN]	39	105	76	39	54	74
m2	Relative ( )	1.59	2.97	1.75	1.40	1.36	1.40

Table 17a

2-bay – integrated connections - deformable diaphragm – four connections per panel.

	INT3/2DEF	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	22	46	76	38	81	142
a2	ratio [%]	0.3	0.6	1.0	0.5	1.1	1.9
b1	Differential top drift [mm]	18	38	58	36	76	128
b2	ratio [%]	0.2	0.5	0.8	0.5	1.0	1.7
c1	Maximum top drift [mm]	22	46	76	38	81	142
c2	Relative ( )	0.23	0.25	0.34	0.39	0.44	0.62
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
f1	Max horizontal force roof-beam [kN]	522	937	1357	775	1476	2379
f2	Relative ( )	6.96	9.76	13.85	10.33	15.21	24.03
g1	Max horizontal force beam-column [kN]	240	404	571	407	885	1521
g2	Relative ( )	3.19	4.21	5.82	5.43	9.12	15.36
h1	Max force panel-beam [kN]	201	321	466	196	317	455
h2	Relative ( )	2.68	3.34	4.76	2.61	3.27	4.60
j1	Total base shear [kN]	4390	8328	12216	2903	5461	9316
j2	Relative ( )	1.88	2.80	4.01	1.25	1.82	3.03
k1	Total column shear [kN]	587	913	1210	751	1090	1506
k2	Relative ( )	0.13	0.11	0.10	0.26	0.20	0.16
l1	Mean column shear [kN]	19	29	39	24	35	49
l2	Relative ( )	0.25	0.31	0.40	0.32	0.36	0.49
m1	Max column shear [kN]	37	53	71	39	55	74
m2	Relative ( )	1.96	1.79	1.81	1.60	1.55	1.53

Table 17b

2-bay – integrated connections - rigid diaphragm – three connections per panel.

	INT3/2RIG	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	18	37	132	36	59	124
a2	ratio [%]	0.2	0.5	1.8	0.5	0.8	1.6
b1	Differential top drift [mm]	6	10	10	26	36	47
b2	ratio [%]	0.1	0.1	0.1	0.3	0.5	0.6
c1	Maximum top drift [mm]	18	37	132	36	59	124
c2	Relative ( )	0.18	0.20	0.58	0.37	0.32	0.54
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
e1	Max force roof-roof [kN]	125	194	186	228	359	447
e2	Relative ( )	1.66	2.02	1.90	3.05	3.70	4.52
f1	Max horizontal force roof-beam [kN]	300	409	400	849	1331	1529
f2	Relative ( )	4.01	4.26	4.08	11.32	13.72	15.44
g1	Max horizontal force beam-column [kN]	227	297	300	808	1196	1578
g2	Relative ( )	3.02	3.10	3.07	10.77	12.33	15.94
h1	Max force panel-beam [kN]	467	671	707	471	758	892
h2	Relative ( )	6.23	6.99	7.21	6.28	7.82	9.01
j1	Total base shear [kN]	6882	10923	13024	5127	8269	9216
j2	Relative ( )	2.94	3.67	4.28	2.21	2.76	3.00
k1	Total column shear [kN]	840	1102	2498	877	1081	2559
k2	Relative ( )	0.12	0.10	0.19	0.17	0.13	0.28
l1	Mean column shear [kN]	27	36	81	28	35	83
l2	Relative ( )	0.36	0.37	0.82	0.38	0.36	0.83
m1	Max column shear [kN]	34	45	83	38	50	93
m2	Relative ( )	1.27	1.26	1.03	1.33	1.43	1.13

Table 18a

2-bay – integrated connections - rigid diaphragm – four connections per panel.

	INT3/2RIG	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	13	22	35	33	61	95
a2	ratio [%]	0.2	0.3	0.5	0.4	0.8	1.3
b1	Differential top drift [mm]	7	10	13	26	42	54
b2	ratio [%]	0.1	0.1	0.2	0.3	0.6	0.7
c1	Maximum top drift [mm]	13	22	35	33	61	95
c2	Relative ( )	0.14	0.12	0.15	0.33	0.33	0.42
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
e1	Max force roof-roof [kN]	141	201	253	259	422	610
e2	Relative ( )	1.88	2.09	2.58	3.46	4.35	6.16
f1	Max horizontal force roof-beam [kN]	319	401	510	958	1515	2214
f2	Relative ( )	4.26	4.17	5.21	12.78	15.62	22.37
g1	Max horizontal force beam-column [kN]	253	328	417	828	1400	1955
g2	Relative ( )	3.38	3.42	4.25	11.04	14.43	19.74
h1	Max force panel-beam [kN]	294	379	551	353	552	818
h2	Relative ( )	3.92	3.95	5.62	4.70	5.69	8.26
j1	Total base shear [kN]	5763	10222	15500	5919	9003	13571
j2	Relative ( )	2.47	3.44	5.09	2.56	3.00	4.41
k1	Total column shear [kN]	579	882	1082	744	993	1212
k2	Relative ( )	0.10	0.09	0.07	0.13	0.11	0.09
l1	Mean column shear [kN]	19	28	35	24	32	39
l2	Relative ( )	0.25	0.30	0.36	0.32	0.33	0.39
m1	Max column shear [kN]	32	39	48	36	44	57
m2	Relative ( )	1.72	1.38	1.39	1.51	1.36	1.45

Table 18b

3-bay – integrated connections - null diaphragm – three connections per panel.

	INT3/3NUL	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	46	82	149	50	88	146
a2	ratio [%]	0.6	1.1	2.0	0.7	1.2	1.9
b1	Differential top drift [mm]	47	85	157	51	88	146
b2	ratio [%]	0.6	1.1	2.1	0.7	1.2	2.0
c1	Maximum top drift [mm]	46	82	149	50	88	146
c2	Relative ( )	0.45	0.42	0.54	0.48	0.45	0.61
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
f1	Max horizontal force roof-beam [kN]	126	128	134	170	239	329
f2	Relative ( )	1.63	1.33	1.36	2.33	2.60	3.35
g1	Max horizontal force beam-column [kN]	137	143	189	198	277	388
g2	Relative ( )	1.78	1.49	1.93	2.71	3.02	3.96
h1	Max force panel-beam [kN]	224	408	671	154	266	416
h2	Relative ( )	2.91	4.25	6.85	2.11	2.89	4.24
j1	Total base shear [kN]	3689	7092	11518	2763	5191	8512
j2	Relative ( )	1.14	1.77	2.80	0.90	1.34	2.06
k1	Total column shear [kN]	1123	1534	2295	1227	1632	2157
k2	Relative ( )	0.30	0.22	0.20	0.44	0.31	0.25
l1	Mean column shear [kN]	27	37	55	29	39	51
l2	Relative ( )	0.35	0.38	0.56	0.40	0.42	0.52
m1	Max column shear [kN]	54	71	100	52	69	93
m2	Relative ( )	2.02	1.94	1.83	1.77	1.79	1.80

Table 19a

3-bay – integrated connections - null diaphragm – four connections per panel.

	INT3/3NUL	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	47	82	153	50	87	150
a2	ratio [%]	0.6	1.1	2.0	0.7	1.2	2.0
b1	Differential top drift [mm]	47	83	158	51	88	150
b2	ratio [%]	0.6	1.1	2.1	0.7	1.2	2.0
c1	Maximum top drift [mm]	47	82	153	50	87	150
c2	Relative ( )	0.45	0.42	0.56	0.48	0.45	0.63
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
f1	Max horizontal force roof-beam [kN]	126	128	132	155	200	279
f2	Relative ( )	1.63	1.33	1.35	2.12	2.17	2.85
g1	Max horizontal force beam-column [kN]	138	144	190	205	298	431
g2	Relative ( )	1.79	1.50	1.93	2.81	3.24	4.40
h1	Max force panel-beam [kN]	138	219	329	132	188	265
h2	Relative ( )	1.79	2.28	3.36	1.81	2.05	2.70
j1	Total base shear [kN]	2960	5732	9882	2681	4831	7632
j2	Relative ( )	0.91	1.43	2.40	0.88	1.24	1.85
k1	Total column shear [kN]	1109	1456	2193	1238	1635	2225
k2	Relative ( )	0.37	0.25	0.22	0.46	0.34	0.29
l1	Mean column shear [kN]	26	35	52	29	39	53
l2	Relative ( )	0.34	0.36	0.53	0.40	0.42	0.54
m1	Max column shear [kN]	54	70	102	52	69	94
m2	Relative ( )	2.06	2.02	1.95	1.76	1.77	1.77

Table 19b

3-bay – integrated connections - deformable diaphragm – three connections per panel.

	INT3/3DEF	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	31	64	105	43	91	157
a2	ratio [%]	0.4	0.9	1.4	0.6	1.2	2.1
b1	Differential top drift [mm]	25	50	79	39	82	139
b2	ratio [%]	0.3	0.7	1.1	0.5	1.1	1.9
c1	Maximum top drift [mm]	31	64	105	43	91	157
c2	Relative ( )	0.30	0.33	0.38	0.42	0.47	0.66
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
f1	Max horizontal force roof-beam [kN]	682	1178	1744	844	1547	2470
f2	Relative ( )	8.85	12.27	17.80	11.56	16.81	25.21
g1	Max horizontal force beam-column [kN]	508	861	1274	460	990	1707
g2	Relative ( )	6.59	8.97	13.00	6.30	10.76	17.42
h1	Max force panel-beam [kN]	293	547	762	240	469	751
h2	Relative ( )	3.80	5.70	7.77	3.28	5.10	7.66
j1	Total base shear [kN]	4887	9050	12986	3916	8067	13862
j2	Relative ( )	1.51	2.26	3.16	1.28	2.08	3.35
k1	Total column shear [kN]	1347	1907	2462	1173	1707	2259
k2	Relative ( )	0.28	0.21	0.19	0.30	0.21	0.16
l1	Mean column shear [kN]	32	45	59	28	41	54
l2	Relative ( )	0.42	0.47	0.60	0.38	0.44	0.55
m1	Max column shear [kN]	44	63	86	43	63	87
m2	Relative ( )	1.38	1.38	1.47	1.55	1.54	1.62

Table 20a

3-bay – integrated connections - deformable diaphragm – four connections per panel.

	INT3/3DEF	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	29	61	102	41	83	145
a2	ratio [%]	0.4	0.8	1.4	0.5	1.1	1.9
b1	Differential top drift [mm]	26	54	88	39	79	135
b2	ratio [%]	0.3	0.7	1.2	0.5	1.0	1.8
c1	Maximum top drift [mm]	29	61	102	41	83	145
c2	Relative ( )	0.28	0.31	0.37	0.39	0.43	0.61
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
f1	Max horizontal force roof-beam [kN]	705	1255	1901	848	1576	2484
f2	Relative ( )	9.16	13.07	19.40	11.62	17.13	25.35
g1	Max horizontal force beam-column [kN]	519	899	1354	442	920	1597
g2	Relative ( )	6.73	9.37	13.82	6.05	10.00	16.29
h1	Max force panel-beam [kN]	183	327	463	185	300	420
h2	Relative ( )	2.37	3.41	4.73	2.53	3.26	4.28
j1	Total base shear [kN]	4416	9260	14270	3754	7308	12878
j2	Relative ( )	1.36	2.31	3.47	1.23	1.88	3.11
k1	Total column shear [kN]	1123	1706	2260	1011	1473	2067
k2	Relative ( )	0.25	0.18	0.16	0.27	0.20	0.16
l1	Mean column shear [kN]	27	41	54	24	35	49
l2	Relative ( )	0.35	0.42	0.55	0.33	0.38	0.50
m1	Max column shear [kN]	43	62	85	43	61	86
m2	Relative ( )	1.60	1.52	1.58	1.78	1.75	1.74

Table 20b

3-bay – integrated connections - rigid diaphragm – three connections per panel.

	INT3/3RIG	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	24	49	105	39	65	122
a2	ratio [%]	0.3	0.7	1.4	0.5	0.9	1.6
b1	Differential top drift [mm]	12	19	19	28	42	49
b2	ratio [%]	0.2	0.2	0.2	0.4	0.6	0.7
c1	Maximum top drift [mm]	24	49	105	39	65	122
c2	Relative ( )	0.23	0.25	0.38	0.37	0.33	0.51
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
e1	Max force roof-roof [kN]	201	318	314	259	414	484
e2	Relative ( )	2.61	3.31	3.20	3.55	4.49	4.94
f1	Max horizontal force roof-beam [kN]	476	646	651	974	1537	1675
f2	Relative ( )	6.18	6.73	6.65	13.34	16.71	17.09
g1	Max horizontal force beam-column [kN]	593	818	806	862	1279	1704
g2	Relative ( )	7.70	8.52	8.22	11.81	13.90	17.39
h1	Max force panel-beam [kN]	533	802	886	468	757	881
h2	Relative ( )	6.92	8.35	9.04	6.42	8.23	8.99
j1	Total base shear [kN]	7967	12488	15184	7250	11976	13344
j2	Relative ( )	2.46	3.11	3.69	2.37	3.09	3.23
k1	Total column shear [kN]	1318	1873	3364	1178	1455	3749
k2	Relative ( )	0.17	0.15	0.22	0.16	0.12	0.28
l1	Mean column shear [kN]	31	45	80	28	35	89
l2	Relative ( )	0.41	0.46	0.82	0.38	0.38	0.91
m1	Max column shear [kN]	40	55	81	40	55	101
m2	Relative ( )	1.27	1.24	1.01	1.42	1.58	1.13

Table 21a

3-bay – integrated connections - rigid diaphragm – four connections per panel.

	INT3/3RIG	y-direction			x-direction		
	Quantity	**0.18g**	**0.36g**	**0.60g**	**0.18g**	**0.36g**	**0.60g**
a1	Maximum top drift [mm]	21	40	64	37	59	94
a2	ratio [%]	0.3	0.5	0.9	0.5	0.8	1.2
b1	Differential top drift [mm]	13	20	26	31	42	56
b2	ratio [%]	0.2	0.3	0.3	0.4	0.6	0.7
c1	Maximum top drift [mm]	21	40	64	37	59	94
c2	Relative ( )	0.20	0.21	0.23	0.35	0.31	0.39
	Max roof-beam relative drift [mm]	0	0	0	0	0	0
e1	Max force roof-roof [kN]	215	334	445	284	439	631
e2	Relative ( )	2.79	3.48	4.54	3.90	4.78	6.43
f1	Max horizontal force roof-beam [kN]	504	677	847	1109	1613	2362
f2	Relative ( )	6.55	7.05	8.65	15.20	17.54	24.11
g1	Max horizontal force beam-column [kN]	612	825	1002	929	1497	2111
g2	Relative ( )	7.95	8.59	10.22	12.73	16.27	21.54
h1	Max force panel-beam [kN]	365	552	799	333	526	791
h2	Relative ( )	4.75	5.75	8.15	4.56	5.71	8.08
j1	Total base shear [kN]	8821	13907	21046	8192	12537	19278
j2	Relative ( )	2.72	3.47	5.12	2.68	3.23	4.66
k1	Total column shear [kN]	1165	1604	2080	1001	1377	1697
k2	Relative ( )	0.13	0.12	0.10	0.12	0.11	0.09
l1	Mean column shear [kN]	28	38	50	24	33	40
l2	Relative ( )	0.36	0.40	0.51	0.33	0.36	0.41
m1	Max column shear [kN]	38	61	65	41	48	64
m2	Relative ( )	1.35	1.59	1.32	1.70	1.45	1.58

Table 21b

Comments

At service limit condition (0.18g), the maximum drifts vary from 0.1% to 0.7% (that is from 4 to 51 mm) for null diaphragm, from 0.1% to 0.6% (that is from 4 to 43 mm) for deformable diaphragm and from 0.1% to 0.5% (that is from 5 to 39 mm) for rigid roof diaphragm. The difference in the maximum drifts for panels connected at three and four points varies up to 0.1%.

At no-collapse limit condition (0.36g), the maximum drifts vary from 0.1% to 1.4% (that is from 10 to 105 mm) for null diaphragm, from 0.1% to 1.2% (that is from 8 to 91 mm) for deformable diaphragm and from 0.1% to 0.9 % (that is from 10 to 65 mm) for rigid roof diaphragm. The difference in the maximum drifts for panels connected at three and four points varies up to 0.2%.

For rigid diaphragm, the horizontal forces induced to roof-to-roof connections vary from 30 kN to 303 kN at service limit conditions (0,18g) and from 36 kN to 497 kN at no-collapse limit conditions (0,36g). The difference in the maximum forces for panels connected at three and four points varies up to 98 kN.

At service limit condition (0.18g), the horizontal forces induced to roof-to-beam connections vary from 78 kN to 170 kN for null diaphragm, from 117 kN to 848 kN for deformable diaphragm and from 115 kN to 1109 kN for rigid roof diaphragm. The difference in the maximum forces for panels connected at three and four points varies up to 251 kN.

At no-collapse limit condition (0.36g), the horizontal forces induced to roof-to-beam connections vary from 95 kN to 239 kN for null diaphragm, from 146 kN to 1576 kN for deformable diaphragm and from 129 kN to 1613 kN for rigid roof diaphragm. The difference in the maximum forces for panels connected at three and four points varies up to 268 kN.

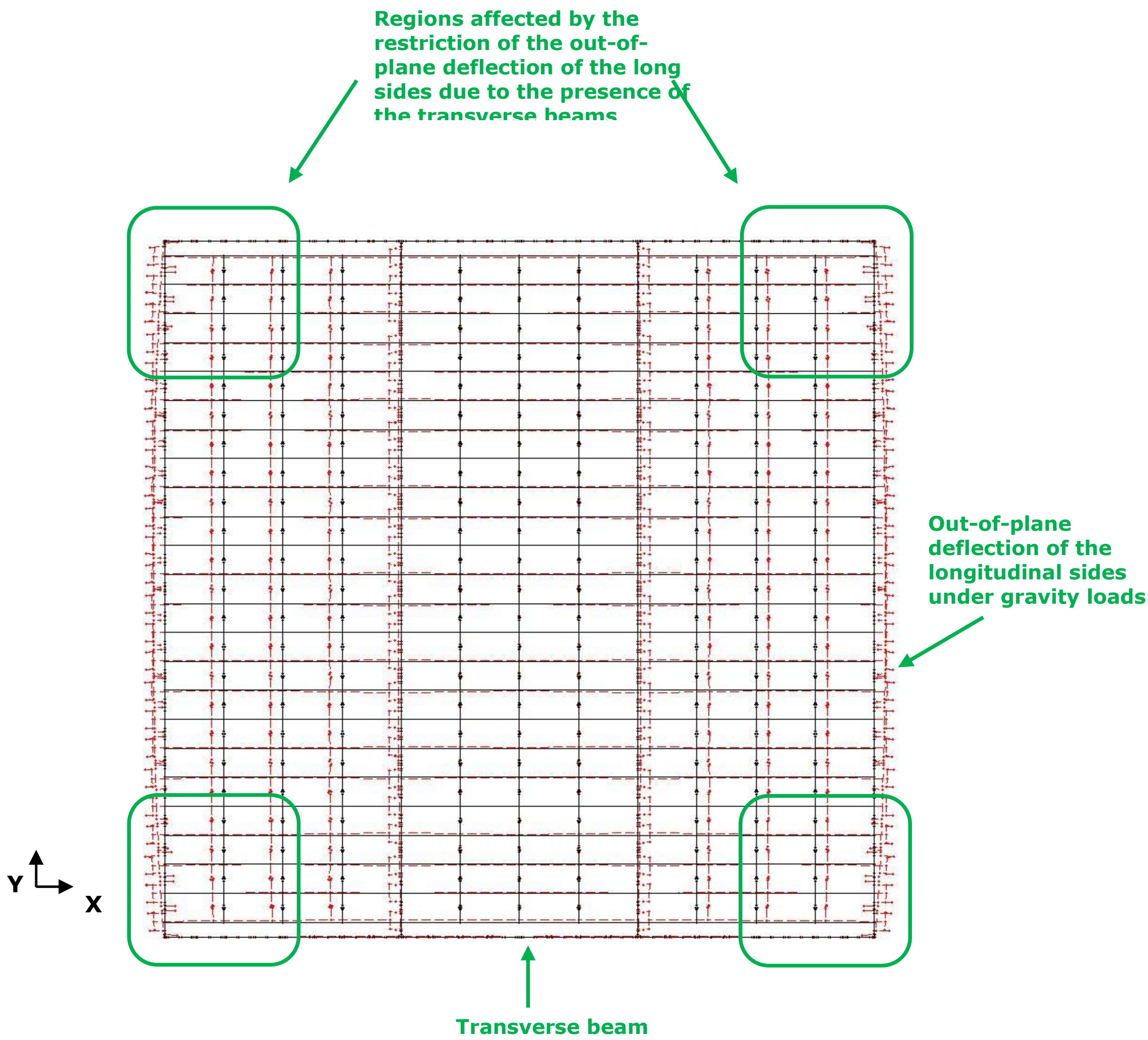
At service limit condition (0.18g), the horizontal forces induced to beam-to-column connections vary from 76 kN to 205 kN for null diaphragm, from 81 kN to 519 kN for deformable diaphragm and from 91 kN to 1045 kN for rigid roof diaphragm. The difference in the maximum forces for panels connected at three and four points varies up to 226 kN.

At no-collapse limit condition (0.36g), the horizontal forces induced to beam-to-column connections vary from 102 kN to 298 kN for null diaphragm, from 100 kN to 990 kN for deformable diaphragm and from 105 kN to 1795 kN for rigid roof diaphragm. The difference in the maximum forces for panels connected at three and four points varies up to 444 kN.

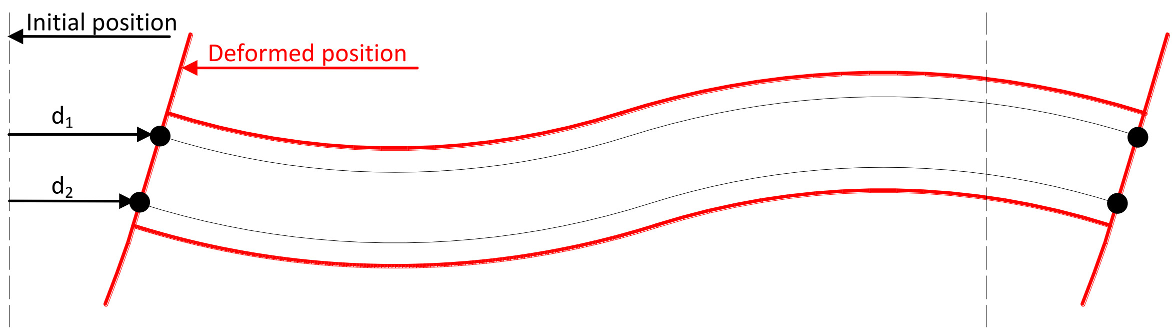
At service limit condition (0.18g), the maximum forces induced to panel-to-beam connections vary from 132 kN to 318 kN for null diaphragm, from 178 kN to 304 kN for deformable diaphragm and from 212 kN to 533 kN for rigid roof diaphragm. The difference in the maximum forces for panels connected at three and four points varies up to 173 kN.

At no-collapse limit condition (0.36g), the maximum forces induced to panel-to-beam connections vary from 188 kN to 408 kN for null diaphragm, from 256 kN to 601 kN for deformable diaphragm and from 287 kN to 946 kN for rigid roof diaphragm. The difference in the maximum forces for panels connected at three and four points varies up to 292 kN.

In some connections, such as the roof-to-roof, roof-to-beam and beam-to-column, there is significant variation of the forces between connections located at different positions. The larger forces develop at the connections located close to the four corners of the building due to the restriction of the horizontal deflection of the longitudinal sides (y direction) of the building at these places caused by the transverse beams. This is shown in Fig. 3a, where the deformed shape of the 3-bay building with rigid roof connection under the gravity loads is depicted. A similar phenomenon is observed for seismic loading, but with displacements developed in the same direction at both sides. It is noted that the transverse beams along the short sides of the building (x direction) are deemed necessary for the construction of the integrated connections of the panels placed on these sides.



a) Deformation of three-bay structure due to gravity loads



b) Deformation of roof elements during seismic loading

Figure 3

Due to the deformation of the transverse beam, the displacements d1 and d2 at the two points where the double-tee roof elements are connected to the beams (at the bottom of their legs) are different from each other (Fig. 3b) resulting in the distortion of the roof elements. Since the double-tees are quite stiff, they cannot accommodate easily this difference in the motion of their supports and, as a result, very large reaction forces develop in the connections (more than 1500 kN for pga=0.36 g and more than 2500 kN for pga=0.60 g in the worst case). It is evident that such large forces cannot develop in reality, as the connections would break.

It is noted that the large forces that develop at the roof-beam connections result in the deformation of the beam as well; therefore, large forces develop at the beam-column connections, too (up to about 1500 kN for pga=0.36 g and 2110 kN for pga=0.60 g in the worst case). Of course, these values correspond to the unrealistic hypothesis that the roof-beam connections will not break. After breaking of the roof-beam connections, the forces at beam-column connections are significantly reduced.

One solution of this problem would be to fasten only one leg of the end roof elements to the beams. In that case, the roof-beam forces would be greatly relaxed, but large forces would develop in the roof-roof connections due to different motion of the adjacent roof elements.

It is mentioned that analyses (not shown here) performed using a tri-linear constitutive law for the roof-beam connections, which takes into account yielding and brittle failure, showed that yielding or breaking of these connections might lead to large displacements of the roof, up to 24 cm for pga=0.36 g. It is recommended, therefore, to apply some kind of protection against falling of the roof, e.g. large seating areas and, if necessary, seismic stoppers similar to the ones used in bridges.

1-bay – plastic dissipative connections – null diaphragm

	DIS3/1NUL	x direction			y direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	144 2,0%	220 3,0%	286 3,9%	160 2,2%	220 3,0%	280 3,8%
b1 *b2*	Differential top drift (mm) Ratio (%)	97 1,3%	142 1,9%	91 1,2%	- -	- -	- -
c1 *c2*	Maximum top drift (mm) Relative ( )	144 1,44	220 1,15	286 1,24	160 1,67	220 1,20	280 1,19
d1 *d2*	^Max connection slide (mm) Relative ( )	0,88 0,009	2,44 0,013	15,06 0,065	1,07 0,011	11,28 0,062	20,56 0,087
e1 *e2*	Max force roof-roof (kN) Relative ( )	- -	- -	- -	- -	- -	- -
f1 *f2*	Max force roof-beam (kN) Relative ( )	1,6 0,02	2,2 0,02	3,4 0,04	2,4 0,03	3,8 0,04	4,6 0,05
g1 *g2*	Max force beam-column (kN) Relative ( )	40,2 0,55	52,6 0,56	68,2 0,70	15,1 0,21	18,1 0,19	22,5 0,23
h1 *h2*	Max force wall-structure (kN) Relative ( )	4,0 0,05	7,9 0,08	11,9 0,12	3,8 0,05	8,3 0,09	12,5 0,13
i1 *i2*	Max force wall-wall (kN) Relative ( )	5,26 0,05	5,26 0,03	5,26 0,02	5,26 0,05	5,26 0,03	5,26 0,02
j1 *j2*	Total base shear (kN) Relative ( )	703 0,48	1114 0,59	1511 0,78	1014 0,71	1322 0,71	1608 0,84
k1 *k2*	Total column shear (kN) Relative ()	597 0,85	920 0,83	1208 0,80	856 0,90	1058 0,80	1369 0,85
l1 *l2*	Mean column shear (kN) Relative ( )	33 0,45	51 0,54	67 0,69	50 0,71	59 0,63	76 0,79
m1 *m2*	Max column shear (kN) Relative ()	35 0,48	47 0,50	61 0,62	48 0,67	58 0,62	68 0,71

Table 22

1-bay – plastic dissipative connections – deformable diaphragm

	DIS3/1DEF	x direction			y direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	99 1,3%	202 2,7%	271 3,7%	76 1,0%	178 2,4%	265 3,6%
b1 *b2*	Differential top drift (mm) Ratio (%)	0,4 0,0%	0,8 0,0%	5,0 0,1%	- -	- -	- -
c1 *c2*	Maximum top drift (mm) Relative ( )	99 0,99	202 1,06	271 1,18	76 0,79	178 0,97	265 1,12
d1 *d2*	^Max connection slide (mm) Relative ( )	2,74 0,027	14,40 0,075	25,73 0,112	2,31 0,024	12,58 0,069	27,79 0,118
e1 *e2*	Max force roof-roof (kN) Relative ( )	6,2 0,08	9,6 0,13	11,4 0,16	5,8 0,08	10,2 0,14	12,9 0,18
f1 *f2*	Max force roof-beam (kN) Relative ( )	12,8 0,18	16,7 0,18	27,7 0,29	19,2 0,27	28,9 0,31	37,5 0,39
g1 *g2*	Max force beam-column (kN) Relative ( )	32 0,44	42 0,45	55 0,56	12 0,17	14 0,16	18 0,19
h1 *h2*	Max force wall-structure (kN) Relative ( )	4,21 0,06	8,82 0,09	13,35 0,14	4,53 0,06	8,51 0,09	14,77 0,15
i1 *i2*	Max force wall-wall (kN) Relative ( )	5,26 0,07	5,26 0,06	5,26 0,05	5,26 0,07	5,26 0,06	5,26 0,05
j1 *j2*	Total base shear (kN) Relative ( )	951 0,65	1233 0,65	1416 0,73	867 0,61	1166 0,62	1436 0,75
k1 *k2*	Total column shear (kN) Relative ()	845 0,89	1127 0,91	1310 0,93	761 0,88	1060 0,91	1330 0,93
l1 *l2*	Mean column shear (kN) Relative ( )	47 0,64	63 0,67	73 0,75	42 0,60	59 0,63	74 0,77
m1 *m2*	Max column shear (kN) Relative ()	49 0,68	58 0,62	66 0,68	40 0,56	58 0,62	66 0,69

Table 23

1-bay – plastic dissipative connections – rigid diaphragm

	DIS3/1RIG	x direction			y direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	83 1,1%	190 2,6%	260 3,5%	52 0,7%	168 2,3%	228 3,1%
b1 *b2*	Differential top drift (mm) Ratio (%)	0,0 0,0%	0,1 0,0%	0,3 0,0%	- -	- -	- -
c1 *c2*	Maximum top drift (mm) Relative ( )	83 0,83	190 1,00	260 1,13	52 0,54	168 0,92	228 0,97
d1 *d2*	^Max connection slide (mm) Relative ( )	3,87 0,039	16,19 0,085	28,99 0,126	1,07 0,011	11,27 0,062	20,59 0,087
e1 *e2*	Max force roof-roof (kN) Relative ( )	11,2 0,15	16,3 0,17	18,2 0,19	10,4 0,15	17,3 0,19	20,6 0,22
f1 *f2*	Max force roof-beam (kN) Relative ( )	21 0,29	26 0,28	31 0,32	23 0,32	35 0,38	39 0,41
g1 *g2*	Max force beam-column (kN) Relative ( )	22 0,30	28 0,30	31 0,32	17 0,24	20 0,22	26 0,27
h1 *h2*	Max force wall-structure (kN) Relative ( )	4,5 0,06	9,4 0,10	14,2 0,15	4,8 0,07	9,0 0,10	15,7 0,16
i1 *i2*	Max force wall-wall (kN) Relative ( )	5,26 0,07	5,26 0,06	5,26 0,05	5,26 0,07	5,26 0,06	5,26 0,05
j1 *j2*	Total base shear (kN) Relative ( )	1012 0,70	1312 0,70	1506 0,78	922 0,65	1240 0,66	1528 0,80
k1 *k2*	Total column shear (kN) Relative ()	906 0,90	1206 0,92	1400 0,93	866 0,89	1134 0,91	1422 0,93
l1 *l2*	Mean column shear (kN) Relative ( )	50,3 0,69	67,0 0,71	77,8 0,80	45,3 0,64	63,0 0,68	79,0 0,82
m1 *m2*	Max column shear (kN) Relative ()	53 0,73	62,2 0,66	70,2 0,72	43 0,61	61,8 0,66	70,6 0,74

Table 24

2-bays – plastic dissipative connections – null diaphragm

		x direction			y direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	130 1,8%	204 2,8%	260 3,5%	146 2,0%	230 3,1%	282 3,8%
b1 *b2*	Differential top drift (mm) Ratio (%)	95 1,3%	123 1,7%	124 1,7%	99 1,3%	140 1,9%	132 1,8%
c1 *c2*	Maximum top drift (mm) Relative ( )	130 1,33	204 1,10	260 1,13	146 1,49	230 1,24	282 1,24
d1 *d2*	^Max connection slide (mm) Relative ( )	0,88 0,009	2,44 0,013	15,06 0,066	1,07 0,011	11,28 0,061	20,56 0,091
e1 *e2*	Max force roof-roof (kN) Relative ( )	- -	- -	- -	- -	- -	- -
f1 *f2*	Max force roof-beam (kN) Relative ( )	1,92 0,03	2,2 0,02	4,25 0,04	2,88 0,04	4,18 0,04	5,52 0,06
g1 *g2*	Max force beam-column (kN) Relative ( )	45,0 0,60	57,9 0,60	71,6 0,72	15,1 0,20	19,9 0,21	23,6 0,24
h1 *h2*	Max force wall-structure (kN) Relative ( )	3,8 0,05	7,6 0,08	11,4 0,12	4,1 0,05	8,0 0,08	12,0 0,12
i1 *i2*	Max force wall-wall (kN) Relative ( )	5,26 0,05	5,26 0,03	5,26 0,02	5,26 0,05	5,26 0,03	5,26 0,02
j1 *j2*	Total base shear (kN) Relative ( )	1510 0,65	1890 0,63	2840 0,92	1220 0,52	1925 0,65	2820 0,93
k1 *k2*	Total column shear (kN) Relative ()	903 0,60	1283 0,68	1720 0,61	805 0,74	1320 0,69	1675 0,59
l1 *l2*	Mean column shear (kN) Relative ( )	33 0,45	48 0,49	64 0,64	34 0,45	49 0,51	62 0,63
m1 *m2*	Max column shear (kN) Relative ()	66 0,88	86 0,89	108 1,09	68 0,91	84 0,88	105 1,07

Table 25

2-bay – plastic dissipative connections – deformable diaphragm

		x-direction			y direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	105 1,4%	196 2,7%	328 4,5%	110 1,5%	210 2,9%	311 4,2%
b1 *b2*	Differential top drift (mm) Ratio (%)	0,6 0,0%	1,0 0,0%	37,7 0,5%	51 0,7%	62 0,8%	71 1,0%
c1 *c2*	Maximum top drift (mm) Relative ( )	105 1,07	196 1,05	328 1,43	110 1,13	210 1,13	311 1,37
d1 *d2*	^Max connection slide (mm) Relative ( )	2,74 0,028	14,40 0,077	25,73 0,112	2,31 0,024	12,58 0,068	27,79 0,122
e1 *e2*	Max force roof-roof (kN) Relative ( )	7,44 0,10	9,6 0,10	14,25 0,14	6,96 0,09	11,22 0,12	15,48 0,16
f1 *f2*	Max force roof-beam (kN) Relative ( )	15,4 0,20	16,7 0,17	34,6 0,35	23,0 0,31	31,8 0,33	45,0 0,46
g1 *g2*	Max force beam-column (kN) Relative ( )	36,0 0,48	46,3 0,48	57,3 0,58	12,1 0,16	15,9 0,17	18,9 0,19
h1 *h2*	Max force wall-structure (kN) Relative ( )	4,3 0,06	8,5 0,09	12,8 0,13	4,6 0,06	8,9 0,09	13,4 0,14
i1 *i2*	Max force wall-wall (kN) Relative ( )	5,26 0,07	5,26 0,06	5,26 0,05	5,26 0,07	5,26 0,06	5,26 0,05
j1 *j2*	Total base shear (kN) Relative ( )	1843 0,80	2092 0,70	2661 0,87	1043 0,45	1697 0,57	2519 0,83
k1 *k2*	Total column shear (kN) Relative ()	1279 0,69	1572 0,75	1865 0,70	758 0,73	1322 0,78	1628 0,65
l1 *l2*	Mean column shear (kN) Relative ( )	47 0,63	58 0,60	69 0,70	28 0,37	49 0,51	60 0,62
m1 *m2*	Max column shear (kN) Relative ()	93 1,25	105 1,09	117 1,18	57 0,76	84 0,88	102 1,04

Table 26

2-bay – plastic dissipative connections – rigid diaphragm

	DIS3/2RIG	x direction			y direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	57 0,8%	162 2,2%	231 3,1%	55 0,7%	120 1,6%	240 3,3%
b1 *b2*	Differential top drift (mm) Ratio (%)	1,7 0,0%	2,7 0,0%	0,2 0,0%	4 0,1%	6 0,1%	11 0,1%
c1 *c2*	Maximum top drift (mm) Relative ( )	57 0,58	162 0,87	231 1,01	55 0,56	120 0,65	240 1,06
d1 *d2*	^Max connection slide (mm) Relative ( )	3,87 0,039	16,19 0,087	28,99 0,127	1,07 0,011	11,27 0,061	20,59 0,091
e1 *e2*	Max force roof-roof (kN) Relative ( )	13,4 0,18	16,3 0,17	22,8 0,23	12,5 0,17	19,1 0,20	24,8 0,25
f1 *f2*	Max force roof-beam (kN) Relative ( )	25,3 0,34	26,4 0,27	38,4 0,39	27,6 0,37	38,8 0,40	47,2 0,48
g1 *g2*	Max force beam-column (kN) Relative ( )	24,6 0,33	30,8 0,32	32,6 0,33	17,0 0,23	22,0 0,23	27,3 0,28
h1 *h2*	Max force wall-structure (kN) Relative ( )	4,8 0,06	9,5 0,10	14,3 0,14	5,1 0,07	10,0 0,10	15,0 0,15
i1 *i2*	Max force wall-wall (kN) Relative ( )	5,26 0,07	5,26 0,06	5,26 0,05	5,26 0,07	5,26 0,06	5,26 0,05
j1 *j2*	Total base shear (kN) Relative ( )	1101 0,44	1312 0,44	1660 0,49	922 0,39	1240 0,42	1564 0,50
k1 *k2*	Total column shear (kN) Relative ()	812 1,09	1288 0,98	1506 1,10	715 0,65	1036 0,84	1378 1,02
l1 *l2*	Mean column shear (kN) Relative ( )	41 0,54	48 0,49	61 0,62	22 0,29	38 0,40	58 0,59
m1 *m2*	Max column shear (kN) Relative ()	52 0,69	56 0,58	65 0,66	27 0,36	50 0,52	56 0,58

Table 27

3-bays – plastic dissipative connections – null diaphragm

	DIS3/3NUL	x direction			y direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	144 2,0%	220 3,0%	286 3,9%	144 2,0%	220 3,0%	214 2,9%
b1 *b2*	Differential top drift (mm) Ratio (%)	103 1,4%	145 2,0%	102 1,4%	97 1,3%	139 1,9%	87 1,2%
c1 *c2*	Maximum top drift (mm) Relative ( )	144 1,38	220 1,13	286 1,20	144 1,39	220 1,12	214 0,78
d1 *d2*	^Max connection slide (mm) Relative ( )	0,88 0,008	2,44 0,013	15,06 0,063	1,07 0,010	11,28 0,058	20,56 0,075
e1 *e2*	Max force roof-roof (kN) Relative ( )	- -	- -	- -	- -	- -	- -
f1 *f2*	Max force roof-beam (kN) Relative ( )	2,30 0,03	2,64 0,03	5,10 0,05	3,5 0,04	5,0 0,05	6,6 0,07
g1 *g2*	Max force beam-column (kN) Relative ( )	50,4 0,69	64,8 0,70	80,2 0,82	15,1 0,20	19,9 0,21	23,6 0,24
h1 *h2*	Max force wall-structure (kN) Relative ( )	3,9 0,05	7,8 0,08	11,7 0,12	3,9 0,05	8,2 0,09	12,2 0,12
i1 *i2*	Max force wall-wall (kN) Relative ( )	5,26 0,05	5,26 0,03	5,26 0,02	5,26 0,05	5,26 0,03	5,26 0,02
j1 *j2*	Total base shear (kN) Relative ( )	2040 0,67	2580 0,66	3060 0,74	1780 0,55	2420 0,60	3040 0,74
k1 *k2*	Total column shear (kN) Relative ()	1570 0,92	1850 0,83	2405 0,79	1405 0,79	1852 0,77	2280 0,75
l1 *l2*	Mean column shear (kN) Relative ( )	52 0,71	60 0,65	67 0,68	39 0,51	51 0,54	63 0,65
m1 *m2*	Max column shear (kN) Relative ()	66 0,90	69 0,75	71 0,72	66 0,86	68 0,71	70 0,71

Table 28

3-bays – plastic dissipative connections – deformable diaphragm

	DIS3/RDEF	x direction			y diretion		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	91 1,2%	194 2,6%	275 3,7%	60 0,8%	92 1,2%	231 3,1%
b1 *b2*	Differential top drift (mm) Ratio (%)	0,7 0,0%	1,1 0,0%	41,5 0,6%	56 0,8%	121 1,6%	78 1,1%
c1 *c2*	Maximum top drift (mm) Relative ( )	91 0,87	194 1,00	275 1,16	60 0,57	92 0,47	231 0,84
d1 *d2*	^Max connection slide (mm) Relative ( )	2,74 0,026	14,40 0,074	25,73 0,108	2,31 0,022	12,58 0,064	27,79 0,101
e1 *e2*	Max force roof-roof (kN) Relative ( )	7,44 0,10	9,6 0,10	14,25 0,15	6,96 0,09	11,22 0,12	15,48 0,16
f1 *f2*	Max force roof-beam (kN) Relative ( )	15,4 0,21	16,7 0,18	34,6 0,35	23,0 0,30	31,8 0,33	45,0 0,46
g1 *g2*	Max force beam-column (kN) Relative ( )	36,0 0,49	46,3 0,50	57,3 0,58	12,1 0,16	15,9 0,17	18,9 0,19
h1 *h2*	Max force wall-structure (kN) Relative ( )	4,2 0,06	8,7 0,09	13,1 0,13	4,6 0,06	8,7 0,09	14,1 0,14
i1 *i2*	Max force wall-wall (kN) Relative ( )	5,26 0,07	5,26 0,06	5,26 0,05	5,26 0,07	5,26 0,06	5,26 0,05
j1 *j2*	Total base shear (kN) Relative ( )	2490 0,81	2856 0,74	3020 0,73	1603 0,47	2134 0,53	3020 0,73
k1 *k2*	Total column shear (kN) Relative ()	1934 0,78	2163 0,76	2420 0,80	1521 1,05	1874 0,93	2430 0,80
l1 *l2*	Mean column shear (kN) Relative ( )	54 0,74	60 0,65	67 0,69	45 0,58	55 0,57	68 0,69
m1 *m2*	Max column shear (kN) Relative ()	61 0,84	66 0,72	68 0,69	62 0,81	67 0,70	70 0,71

Table 29

3-bays – plastic dissipative connections – rigid diaphragm

		x direction			y direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	65 0,9%	178 2,4%	251 3,4%	57 0,8%	96 1,3%	243 3,3%
b1 *b2*	Differential top drift (mm) Ratio (%)	1,3 0,0%	1,6 0,0%	6,4 0,1%	3 0,0%	5 0,1%	10 0,1%
c1 *c2*	Maximum top drift (mm) Relative ( )	65 0,63	178 0,92	251 1,05	57 0,54	96 0,49	243 0,88
d1 *d2*	^Max connection slide (mm) Relative ( )	3,87 0,037	16,19 0,083	28,99 0,122	1,07 0,010	11,27 0,057	20,59 0,075
e1 *e2*	Max force roof-roof (kN) Relative ( )	13,4 0,18	16,3 0,18	22,8 0,23	12,5 0,16	19,1 0,20	24,8 0,25
f1 *f2*	Max force roof-beam (kN) Relative ( )	25,3 0,35	26,4 0,29	38,4 0,39	27,6 0,36	38,8 0,40	47,2 0,48
g1 *g2*	Max force beam-column (kN) Relative ( )	24,6 0,34	30,8 0,33	32,6 0,33	17,0 0,22	22,0 0,23	27,3 0,28
h1 *h2*	Max force wall-structure (kN) Relative ( )	4,6 0,06	9,4 0,10	14,2 0,15	5,0 0,06	9,5 0,10	15,3 0,16
i1 *i2*	Max force wall-wall (kN) Relative ( )	5,26 0,07	5,26 0,06	5,26 0,05	5,26 0,07	5,26 0,06	5,26 0,05
j1 *j2*	Total base shear (kN) Relative ( )	2005 0,66	2551 0,66	3102 0,75	1728 0,51	2485 0,62	3220 0,78
k1 *k2*	Total column shear (kN) Relative ()	1557 0,78	1932 0,76	2486 0,80	1640 1,05	2299 0,93	2591 0,80
l1 *l2*	Mean column shear (kN) Relative ( )	43 0,59	54 0,58	69 0,70	48 0,62	64 0,67	72 0,73
m1 *m2*	Max column shear (kN) Relative ()	49 0,68	59 0,64	71 0,72	67 0,87	78 0,81	75 0,76

Table 30

Comments

At the service level (0.18g PGA), the drift ratios vary between 0.7 to 2.2%. The drift ratio decreases significantly in the case of rigid diaphragm.

At the design level (0.36g PGA), the drifts vary between 1.2 to 3.7%.

At the maximum earthquake level (0.60g PGA), the top drifts vary between 2.9 to 4.5%. It should be noted that the drift ratios are quite high as compared to those would be experienced in a residential or office building. The reason for that is first the columns examined here are under single-bending action, and secondly, the columns are quite slender with low axial load ratio on top leading thus to capability to sustain higher drift ratios.

The ratio of the base shear born by the panels is in the order of 7 to 11% for the single-bay structures. The same parameter is in the order of 20-22% in 3-bay structures when the diaphragm is rigid and decreases to 8-15% when the diaphragm action is null. The effectiveness of the panels increases significantly as the diaphragm action becomes more pronounced.

The roof-to-beam connection forces are in the order of 26 to 40kN per connection in the case of 3-bay structure with rigid diaphragm. These values are already high and are expected to be higher in case the effectiveness of the panels increase. There is a trend that the roof-to-beam and roof-to-roof connection forces increase, expectedly, as the fraction of the base shear force carried by the panels increase. This issue has to be taken care of during the design phase.

The differential drift, between the edge frame with attached panels and the mid-frame that is bare, is in the order of 2.0% that is quite high. High differential drift values are not acceptable due to stability concerns, and it is directly correlated with the effectiveness of the panel connections. The differential excitation of the frames in a certain earthquake direction is inevitable in case of loose diaphragm action when the stiffness and strength of the frames with attached panels is much different than that of the bare frames.

General Comments

The base shear contribution of the panels did not exceed 22% in any case, a value that is low for the proper energy dissipation. A quick estimate of the equivalent damping based on the base shear contribution of the bare frame and of the panel, also assuming a 5% damping for the RC frame and 15% damping for the steel dissipative connections, provides us that:

$$\xi_{eq} = 0{,}05(1 - 0{,}22) + 0{,}15(0{,}22) = 0{,}07$$

This increases new equivalent damping would decrease the seismic forces acting on the system:

$$\eta = \sqrt{\frac{7}{2 + \xi_{eq}}} = 0{,}88$$

only 12%. In order to increase the effectiveness of the panels and use efficiently the plastic connectors, the contribution of the panels has to be increased, and the diaphragm action has to be assured. Various different panel-dissipator-structure connections can be studied to increase the efficiency of the connections.

1-bay – friction dissipative connections – null diaphragm

	DIS3-1nul	x - direction			y - direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	111 1,5	218 2,9	346 4,6	6 0,1	21 0,3	38 0,5
b1 *b2*	Differential top drift (mm) Ratio (%)	113 1,5	218 2,9	375 5,0	74 1,0	132 1,8	175 2,3
c1 *c2*	Maximum top drift (mm) Relative ( )	111 1.11	218 1.14	346 1.73	6 0.07	21 1.11	38 1.16
d1 *d2*	^Max connection slide (mm) Relative ( )	1 0.01	5 0.03	11 0.05	2 0.02	6 0.03	11 0.05
f1 *f2*	Max force roof-beam (kN) Relative ( )	35 0.48	50 0.53	72 0.74	21 0.29	22 0.24	29 0.31
g1 *g2*	Max force beam-column (kN) Relative ( )	68 0.93	90 0.95	101 1.05	21 0.30	39 0.42	60 0.62
h1 *h2*	Max force wall-structure (kN) Relative ( )	300 4.11	311 3.31	319 3.29	309 4.36	315 3.38	326 3.39
i1 *i2*	Max force wall-wall (kN) Relative ( )	60 0.82	60 0.64	60 0.62	60 0.85	60 0.65	60 0.62
j1 *j2*	Total base shear (kN) Relative ( )	2300 1.58	4226 2.24	5622 2.90	3837 2.70	4706 2.52	5398 2.83
k1 *k2*	Total column shear (kN) Relative ( )	5734 0.25	934 0.22	1174 0.21	387 0.10	406 0.09	656 0.12
l1 *l2*	Mean column shear (kN) Relative ( )	29 0.39	47 0.50	59 0.61	19 0.27	20 0.22	33 0.34
m1 *m2*	Max column shear (kN) Relative ( )	76 2.65	111 2.37	122 2.07	59 3.03	45 2.21	69 2.11
h1v *h1h*	Max vert.force wall-struct.(kN) Max hor. force wall-struct.(kN)	297,98 66,18	308,17 74,19	314,71 83,35	306,66 67,29	311,59 73,10	321,56 83,99

^ connection slides all equal Table 31

1-bay – friction dissipative connections – deformable diaphragm

	DIS3-1def	x - direction			y - direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	45 0,6	80 1,1	124 1,6	6 0,1	18 0,2	40 0,5
b1 *b2*	Differential top drift (mm) Ratio (%)	46 0,6	67 0,9	66 0,9	74 1,0	137 1,8	165 2,2
c1 *c2*	Maximum top drift (mm) Relative ( )	45 0,45	80 0,42	124 0,54	6 0,06	18 0,10	40 0.17
d1 *d2*	^Max connection slide (mm) Relative ( )	4 0,04	14 0,08	31 0,14	2 0,03	5 0,03	12 0,05
f1 *f2*	Max force roof-beam (kN) Relative ( )	517 7.08	735 7.82	715 7,37	73 1,03	87 0,93	147 1.53
g1 *g2*	Max force beam-column (kN) Relative ( )	79 1,08	113 1,20	129 1,33	23 0,32	48 0,51	95 1,00
h1 *h2*	Max force wall-structure (kN) Relative ( )	309 4,23	315 3,35	328 3,38	312 4,39	316 3,39	334 3,48
i1 *i2*	Max force wall-wall (kN) Relative ( )	60,00 0,82	60,00 0,64	60,00 0,62	60,00 0,85	60,00 0,65	60,00 0,62
j1 *j2*	Total base shear (kN) Relative ( )	2435 1,68	3554 1,88	3890 2,01	3694 2,60	4930 2,64	5265 2,76
k1 *k2*	Total column shear (kN) Relative ( )	824 0,34	1218 0,34	2873 0,734	415 0,11	759 0,15	813 0,15
l1 *l2*	Mean column shear (kN) Relative ( )	41 0,56	61 0,65	144 1,48	21 0,29	38 0,41	41 0,42
m1 *m2*	Max column shear (kN) Relative ( )	167 4,05	149 2,44	204 1,42	33 1,62	59 1,56	92 2,27
h1v *h1h*	Max vert.force wall-struct.(kN) Max hor. force wall-struct.(kN)	307 66	312 63	323 85	310 67	313 75	330 83

^ connection slides all equal Table 32

1-bay – friction dissipative connections – rigid diaphragm

	DIS3-1rig	x - direction			y - direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	32 0,4	62 0,8	113 1,5	9 0,1	30 0,4	73 1,0
b1 *b2*	Differential top drift (mm) Ratio (%)	7 0,1	8 0,1	10 0,1	0 0,0	0 0,0	0 0,0
c1 *c2*	Maximum top drift (mm) Relative ( )	32 0,32	62 0,32	113 1,17	9 0,09	30 0,16	73 0,31
d1 *d2*	^Max connection slide (mm) Relative ( )	9 0,09	18 0,10	34 0,15	3 0,03	10 0,05	22 0,09
e1 *e2*	Max force roof-roof (kN) Relative ( )	196,40 2,69	227,06 2,42	328,84 3,39	132,37 1,86	153,72 1,65	183,49 1,91
f1 *f2*	Max force roof-beam (kN) Relative ( )	31 0,42	43 0,46	43 0,45	52 0,73	56 0,60	63 0,66
g1 *g2*	Max force beam-column (kN) Relative ( )	59 0,81	91 0,97	136 1,41	12 0,17	30 0,32	59 0,61
h1 *h2*	Max force wall-structure (kN) Relative ( )	309 4,23	320 3,44	327 3,51	313 3,36	315 3,39	324 3,37
i1 *i2*	Max force wall-wall (kN) Relative ( )	60,00 0,82	60,00 0,64	60,00 0,62	60,00 0,85	60,00 0,65	60,00 0,62
j1 *j2*	Total base shear (kN) Relative ( )	2051 1,41	2978 1,58	3958 2,04	3879 2,73	4800 2,57	5349 2,80
k1 *k2*	Total column shear (kN) Relative ( )	959 0,47	1589 0,53	2423 0,61	334 0,09	621 0,13	1038 0,19
l1 *l2*	Mean column shear (kN) Relative ( )	53 0,73	88 0,94	135 1,39	19 0,26	35 0,37	58 0,60
m1 *m2*	Max column shear (kN) Relative ( )	59 1,10	96 1,08	145 1,08	19 1,04	36 1,04	75 1,29
h1v *h1h*	Max vert.force wall-struct.(kN) Max hor. force wall-struct.(kN)	307,17 59,04	317,24 69,39	323,24 74,66	307,97 66,06	309,62 74,83	316,10 83,73

^ connection slides all equal   Table 33

2-bays – friction dissipative connections – null diaphragm

	DIS3-2nul	x - direction			y - direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	114 1,5	224 3,0	321 4,3	95 1,3	209 2,8	328 4,4
b1 *b2*	Differential top drift (mm) Ratio (%)	113 1,5	227 3,0	318 4,2	94 1,3	204 2,7	335 4,5
c1 *c2*	Maximum top drift (mm) Relative ( )	114 1,16	224 1,21	321 1,40	95 0,97	209 1,12	328 1,45
d1 *d2*	^Max connection slide (mm) Relative ( )	1 0,01	3 0,01	5 0,04	2 0,02	7 0,04	16 0,07
f1 *f2*	Max force roof-beam (kN) Relative ( )	5 0,07	11 0,11	15 0,15	39 0,52	48 0,50	70 0,71
g1 *g2*	Max force beam-column (kN) Relative ( )	83 1,11	99 1,02	122 1,23	93 1,24	133 1,39	166 1,69
h1 *h2*	Max force wall-structure (kN) Relative ( )	244 3,25	311 3,20	319 3,22	310 4,13	316 3,30	324 3,31
i1 *i2*	Max force wall-wall (kN) Relative ( )	57 0,76	60 0,62	60 0,61	60 0,80	60 0,62	60 0,61
j1 *j2*	Total base shear (kN) Relative ( )	2590 1,12	3927 1,31	5925 1,93	4043 1,73	5145 1,73	6269 2,06
k1 *k2*	Total column shear (kN) Relative ( )	1485 0,57	2447 0,62	3029 0,51	1045 0,26	1322 0,26	1658 0,26
l1 *l2*	Mean column shear (kN) Relative ( )	48 0,64	79 0,81	98 0,99	34 0,45	43 0,44	53 0,55
m1 *m2*	Max column shear (kN) Relative ( )	78 1,63	114 1,45	129 1,32	95 2,83	137 3,21	230 4,30
h1v *h1h*	Max vert.force wall-struct.(kN) Max hor. force wall-struct.(kN)	241 59	307 74	314 83	308 68	313 75	320 83

^ connection slides all equal Table 34

2-bays – friction dissipative connections – deformable diaphragm

	DIS3-2def	x - direction			y - direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	69 0,9	120 1,6	133 1,8	41 0,5	54 0,7	82 1,1
b1 *b2*	Differential top drift (mm) Ratio (%)	46 0,6	67 0,9	66 0,9	38 0,5	45 0,6	54 0,7
c1 *c2*	Maximum top drift (mm) Relative ( )	69 0,71	120 0,65	133 0,58	41 0,42	54 0,29	82 0,36
d1 *d2*	^Max connection slide (mm) Relative ( )	3 0,03	11 0,06	29 0,13	2 0,01	9 0,05	23 0,10
f1 *f2*	Max force roof-beam (kN) Relative ( )	- -	- -	1007 10,17	362 4,82	- -	541 5,52-
g1 *g2*	Max force beam-column (kN) Relative ( )	- -	- -	163 1,65	38 0,51	- -	66 0,67
h1 *h2*	Max force wall-structure (kN) Relative ( )	308,39 4,11	317,34 3,27	327,34 3,31	312,50 4,17	315,77 3,29	331,67 3,38
i1 *i2*	Max force wall-wall (kN) Relative ( )	60 0,80	60 0,62	60 0,61	60 0,80	60 0,62	60 0,61
j1 *j2*	Total base shear (kN) Relative ( )	2345 1,01	3426 1,14	4391 1,43	4086 1,75	5585 1,88	6594 2,17
k1 *k2*	Total column shear (kN) Relative ( )	2246 0,96	3444 1,01	4117 0,94	825 0,20	1149 0,20	1714 0,26
2 2	Mean column shear (kN) Relative ( )	72 0,97	111 1,15	133 1,34	27 0,35	37 0,39	55 0,56
m1 *m2*	Max column shear (kN) Relative ( )	121 1,67	163 1,47	192 1,44	65 2,43	81 2,18	124 2,24
h1v *h1h*	Max vert.force wall-struct.(kN) Max hor. force wall-struct.(kN)	306 67	314 74	323 83	310 67	317 75	328 84

^ connection slides all equal Table 35

2-bays – friction dissipative connections – rigid diaphragm

	DIS3-2rig	x - direction			y - direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	26 0,3	57 0,8	106 1,4	17 0,2	50 0,7	93 1,2
b1 *b2*	Differential top drift (mm) Ratio (%)	5 0,1	7 0,1	7 0,1	3 0,0	5 0,1	4 0,1
c1 *c2*	Maximum top drift (mm) Relative ( )	26 0,27	57 0,31	106 0,46	17 0,17	50 0,27	93 0,41
d1 *d2*	^Max connection slide (mm) Relative ( )	6,61 0,07	14,83 0,08	28,95 0,13	4,14 0,04	6,80 0,04	26,66 0,12
e1 *e2*	Max force roof-roof (kN) Relative ( )	309 4,13	354 3,65	383 3,87	151 2,02	235 2,45	208 2,13
f1 *f2*	Max force roof-beam (kN) Relative ( )	58,46 0,78	67,07 0,69	70,74 0,71	69,13 0,92	93,77 0,98	87,50 0,89
g1 *g2*	Max force beam-column (kN) Relative ( )	48 0,64	88 0,91	138 1,40	23 0,30	46 0,48	73 0,74
h1 *h2*	Max force wall-structure (kN) Relative ( )	311 4,15	320 3,30	324 3,27	311 4,15	321 3,34	329 3,36
i1 *i2*	Max force wall-wall (kN) Relative ( )	60 0,80	60 0,62	60 0,61	60 0,80	60 0,62	60 0,61
j1 *j2*	Total base shear (kN) Relative ( )	2950 1,27	4147 1,38	5391 1,75	4586 1,96	5638 2,00	5969 1,96
k1 *k2*	Total column shear (kN) Relative ( )	1419 0,48	2337 0,56	3350 0,62	665 0,14	1603 0,28	1874 0,31
l1 *l2*	Mean column shear (kN) Relative ( )	53 0,70	87 1,15	124 1,25	25 0,33	59 0,62	69 0,71
m1 *m2*	Max column shear (kN) Relative ( )	54 1,03	93 1,07	143 1,16	29 1,19	76 1,27	107 1,55
h1v *h1h*	Max vert.force wall-struct.(kN) Max hor. force wall-struct.(kN)	309 61	318 66	320 76	307 67	315 72	322 86

^ connection slides all equal Table 36

3-bays – friction dissipative connections – null diaphragm

	DIS3-3nul	x - direction			y - direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	116 1,5	200 2,7	315 4,2	96 1,3	201 2,7	313 4,2
b1 *b2*	Differential top drift (mm) Ratio (%)	115 1,5	199 2,6	316 4,2	98 1,3	202 2,7	320 4,3
c1 *c2*	Maximum top drift (mm) Relative ( )	116 1,11	200 1,03	315 1,32	96 0,92	201 1,03	313 1,14
d1 *d2*	^Max connection slide (mm) Relative ( )	1 0,01	1 0,01	3 0,01	2 0,02	5 0,03	13 0,05
f1 *f2*	Max force roof-beam (kN) Relative ( )	93 1,28	169 1,84	230 2,34	124 1,60	212 2,20	373 3,81
g1 *g2*	Max force beam-column (kN) Relative ( )	83,15 1,14	103,03 1,12	125,43 1,28	68,67 0,89	94,97 0,99	102,67 1,05
h1 *h2*	Max force wall-structure (kN) Relative ( )	203 2,78	306 3,32	316 3,23	312 4,05	312 3,25	319 3,25
i1 *i2*	Max force wall-wall (kN) Relative ( )	33 0,45	59 0,64	60 0,61	60 0,78	60 0,62	60 0,61
j1 *j2*	Total base shear (kN) Relative ( )	3378 1,10	6531 1,68	8871 2,14	4949 1,53	6051 1,51	8450 2,06
k1 *k2*	Total column shear (kN) Relative ( )	1247 0,37	1749 0,27	2378 0,27	1425 0,29	1850 0,31	2174 0,26
l1 *l2*	Mean column shear (kN) Relative ( )	30 0,41	42 0,45	57 0,58	34 0,44	44 0,46	52 0,53
m1 *m2*	Max column shear (kN) Relative ( )	81 2,72	116 2,78	133 2,35	82 2,41	117 2,65	152 2,93
h1v *h1h*	Max vert.force wall-struct.(kN) Max hor. force wall-struct.(kN)	201 26	302 50	311 66	310 67	309 74	314 82

^ connection slides all equal Table 37

3-bays – friction dissipative connections – deformable diaphragm

	DIS3-3def	x - direction			y - direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	78 1,0	111 1,5	161 2,1	56 0,7	76 1,0	119 1,6
b1 *b2*	Differential top drift (mm) Ratio (%)	80 1,1	110 1,5	121 1,6	46 0,6	76 1,0	80 1,1
c1 *c2*	Maximum top drift (mm) Relative ( )	78 0,75	111 0,57	161 0,67	56 0,53	76 0,39	119 0,43
d1 *d2*	^Max connection slide (mm) Relative ( )	2 0,02	10 0,05	31 0,13	3 0,03	10 0,05	34 0,12
f1 *f2*	Max force roof-beam (kN) Relative ( )	319 4,37	423 4,60	538 5,49	354 4,60	497 5,17	559 5,70
g1 *g2*	Max force beam-column (kN) Relative ( )	84 1,15	125 1,36	145 1,48	54 0,70	55 0,58	83 0,85
h1 *h2*	Max force wall-structure (kN) Relative ( )	314,39 4,31	321,96 3,50	324,95 3,32	312,37 4,06	313,39 3,26	343,17 3,50
i1 *i2*	Max force wall-wall (kN) Relative ( )	60,00 0,82	60,00 0,65	60,00 0,61	60,00 0,78	60,00 0,62	60,00 0,61
j1 *j2*	Total base shear (kN) Relative ( )	5275 1,72	7164 1,85	9873 2,39	5594 1,35	5964 1,49	8042 1,96
k1 *k2*	Total column shear (kN) Relative ( )	1776 0,34	3144 0,44	5002 0,51	1116 0,20	1804 0,30	3288 0,41
l1 *l2*	Mean column shear (kN) Relative ( )	42 0,58	75 0,81	119 1,22	27 0,35	43 0,45	78 0,80
m1 *m2*	Max column shear (kN) Relative ( )	113 2,68	146 1,95	169 1,42	47 1,76	69 1,60	95 1,21
h1v *h1h*	Max vert.force wall-struct.(kN) Max hor. force wall-struct.(kN)	312 65	319 48	320 85	310 66	310 74	339 83

^ connection slides all equal Table 38

3-bays – friction dissipative connections – rigid diaphragm

	DIS3-3rig	x - direction			y - direction		
		0,18g	**0,36g**	**0,60g**	**0,18g**	**0,36g**	**0,60g**
a1 *a2*	Maximum top drift (mm) Ratio (%)	29 0,4	60 0,8	110 1,5	22 0,3	55 0,7	98 1,3
b1 *b2*	Differential top drift (mm) Ratio (%)	5 0,1	7 0,1	7 0,1	6 0,10	6 0,1	7 0,1
c1 *c2*	Maximum top drift (mm) Relative ( )	29 0,27	60 0,31	110 0,46	22 0,22	56 0,28	96 0,35
d1 *d2*	^Max connection slide (mm) Relative ( )	7 0,07	15 0,08	30 0,13	6 0,06	15 0,08	30 0.11
e1 *e2*	Max force roof-roof (kN) Relative ( )	229 3,14	252 2,74	265 3,44	285 3,70	285 2,96	293 2,99
f1 *f2*	Max force roof-beam (kN) Relative ( )	65,39 0,90	68,13 0,74	88,31 0,90	76,83 1,00	88,45 0,92	104,74 1,07
g1 *g2*	Max force beam-column (kN) Relative ( )	39,02 0,53	75,06 0,82	121,23 1,24	25,42 0,33	43,80 0,46	75,45 0,77
h1 *h2*	Max force wall-structure (kN) Relative ( )	328 4,50	329 3,58	335 3,42	314 4,07	319 3,33	326 3,33
i1 *i2*	Max force wall-wall (kN) Relative ( )	60 0,82	60 0,65	60 0,61	60 0,78	60 0,62	60 0,61
j1 *j2*	Total base shear (kN) Relative ( )	4892 1,60	6316 1,63	8928 2,16	4849 1,50	6635 1,65	6777 1,65
k1 *k2*	Total column shear (kN) Relative ( )	2279 0,47	4236 0,67	6081 0,68	1392 0,29	2132 0,32	3324 0,49
l1 *l2*	Mean column shear (kN) Relative ( )	63 0,87	118 1,28	169 1,72	39 0,50	59 0,62	92 0,94
m1 *m2*	Max column shear (kN) Relative ( )	87 1,37	169 1,44	248 1,47	63 1,63	90 1,52	148 1,60
h1v *h1h*	Max vert.force wall-struct.(kN) Max hor. force wall-struct.(kN)	326 73	326 79	330 87	309 66	314 73	319 83

^ connection slides all equal Table 39

Comments

At service (0,18g) limit conditions, maximum drifts varying from 1,5% to 1,0% and 0,4% have been evaluated respectively for null, deformable and rigid diaphragm (that is from 116 to 78 and to 29 mm).

At no-collapse (0,36g) limit conditions, maximum drifts varying from 3,0% to 1,6% and 0,8% have been evaluated respectively for null, deformable and rigid diaphragm (that is from 224 to 120 and to 57 mm).

With a rigid diaphragm, in roof-to-roof connections maximum forces from 196 kN to 309 kN have been evaluated at service (0,18g) limit conditions and from 227 to 354 at no-collapse (0,36g) limit conditions.

At service (0,18g) limit conditions, in roof-to-beam connections maximum forces from 35 kN to 124 kN have been evaluated for null and rigid diaphragm, from 354 kN to 517 kN for deformable diaphragm.

At no-collapse (0,36g) limit conditions, in roof-to-beam connections maximum forces from 35 kN to 124 kN have been evaluated for null and rigid diaphragm, from 497 kN to 735 kN for deformable diaphragm.

In general the forces in roof-to-roof and roof-to-beam connections are really high especially for deformable diaphragm and this causes problems for their design.

At service (0,18g) limit conditions, in beam-to-column connections maximum forces from 39 kN to 84 kN have been evaluated for all types of diaphragm.

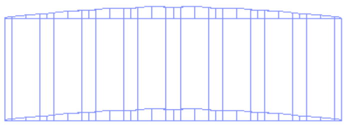
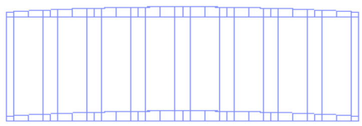
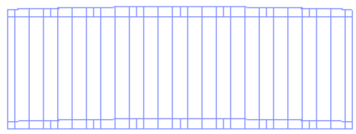
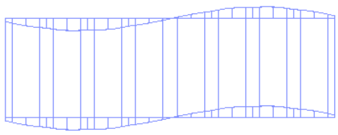
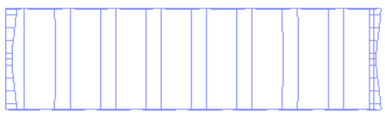
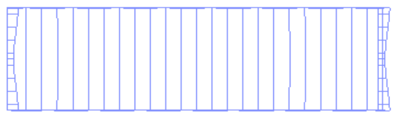
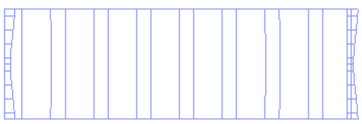
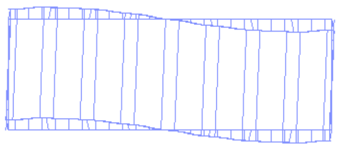
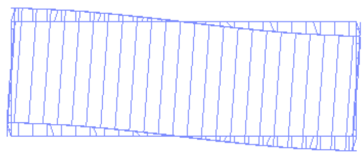
At no-collapse (0,36g) limit conditions, in beam-to-column connections maximum forces from 75 kN to 125 kN have been evaluated for all types of rigid diaphragm.

At both in service (0,18g) and no-collapse (0,36g) limit conditions, an almost constant value of the force (from 309 to 328 kN) in the panel-to-structure connections derives from the slide limit of friction devices.

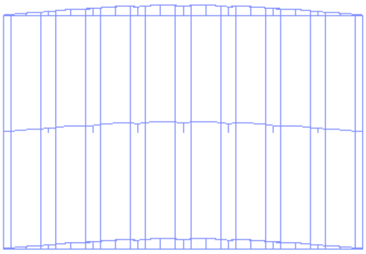
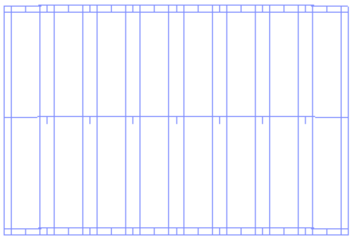
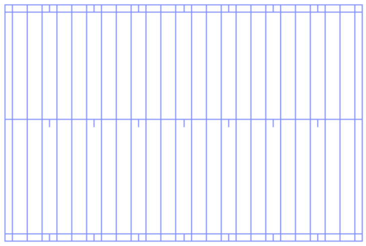
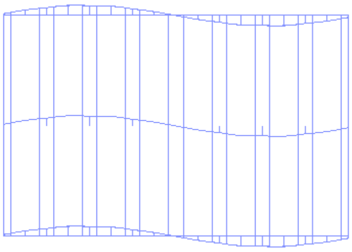
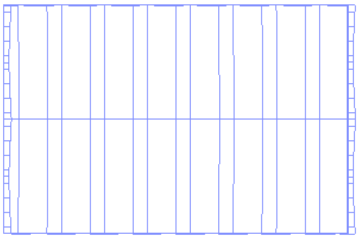
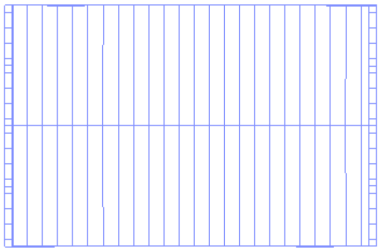
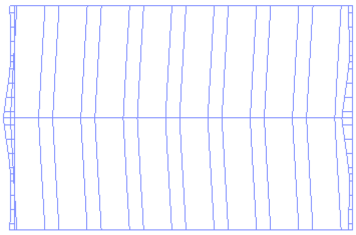
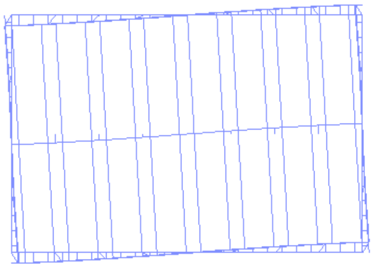
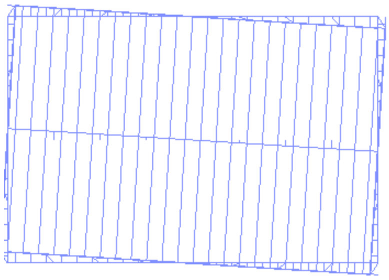
ISOSTATIC SYSTEM OF CONNECTIONS

The plan view of the first three modes of vibration of all analyzed buildings with vertical panels is presented in Figures 16-18. The mode shapes of the structures with horizontal panels are pretty much the same. The reason is that the panels are isolated from the structure and do not contribute to the stiffness of the whole structure. However, there is some slight difference due to the different contributing mass of the panels.

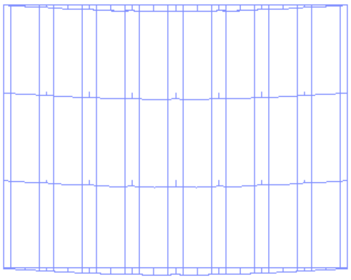
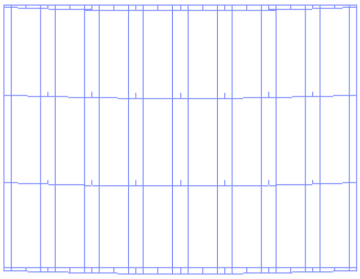
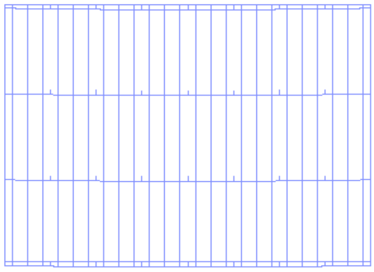
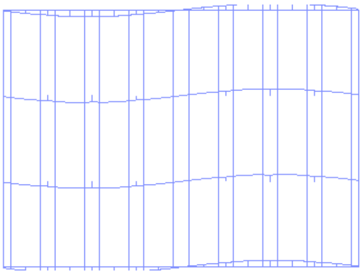
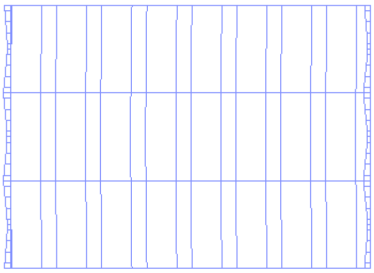
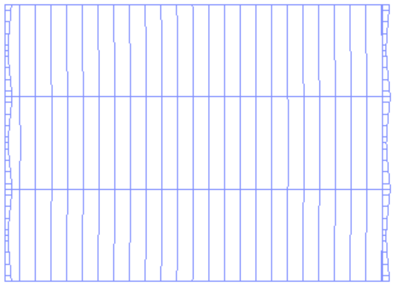
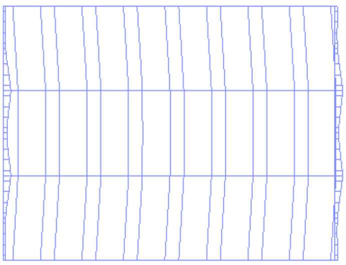
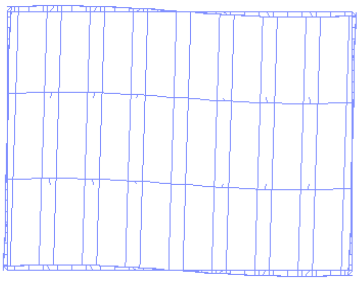
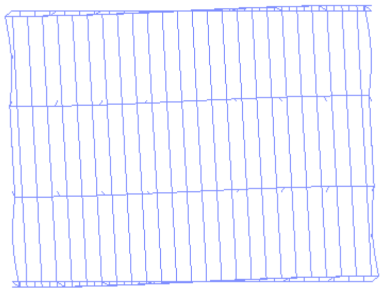
In the case of deformable and rigid diaphragm, the first two modes were translational, while the third was torsional. For the null diaphragm, the second mode was torsional. In general, quite similar modal properties observed for the deformable and rigid diaphragm for 1, 2 and 3-bay structures. The roof beams acted as a rigid diaphragm, since their stiffness was large enough comparing to the very flexible cantilever columns, which represented the main structural system supporting the seismic load. Consequently, the uniform displacements at the roof level were observed. This was not the case for the structures with null diaphragm. Consequently the periods of these structures are longer.

1 bay; null diaphragm	**1 bay; deformable diaphragm**	**1 bay; rigid diaphragm**
T1 = 0.83 sec: transverse dir. 	T1 = 0.77 sec: transverse dir. 	T1 =0.75 sec: transverse dir. 
T2 = 0.78 sec: torsional 	T2 = 0.69 sec: longitudinal dir. 	T2 =0.69 sec: longitudinal dir. 
T3 = 0.69 sec: longitudinal dir. 	T3 = 0.65 sec: torsional 	T3 = 0.65 sec: torsional 

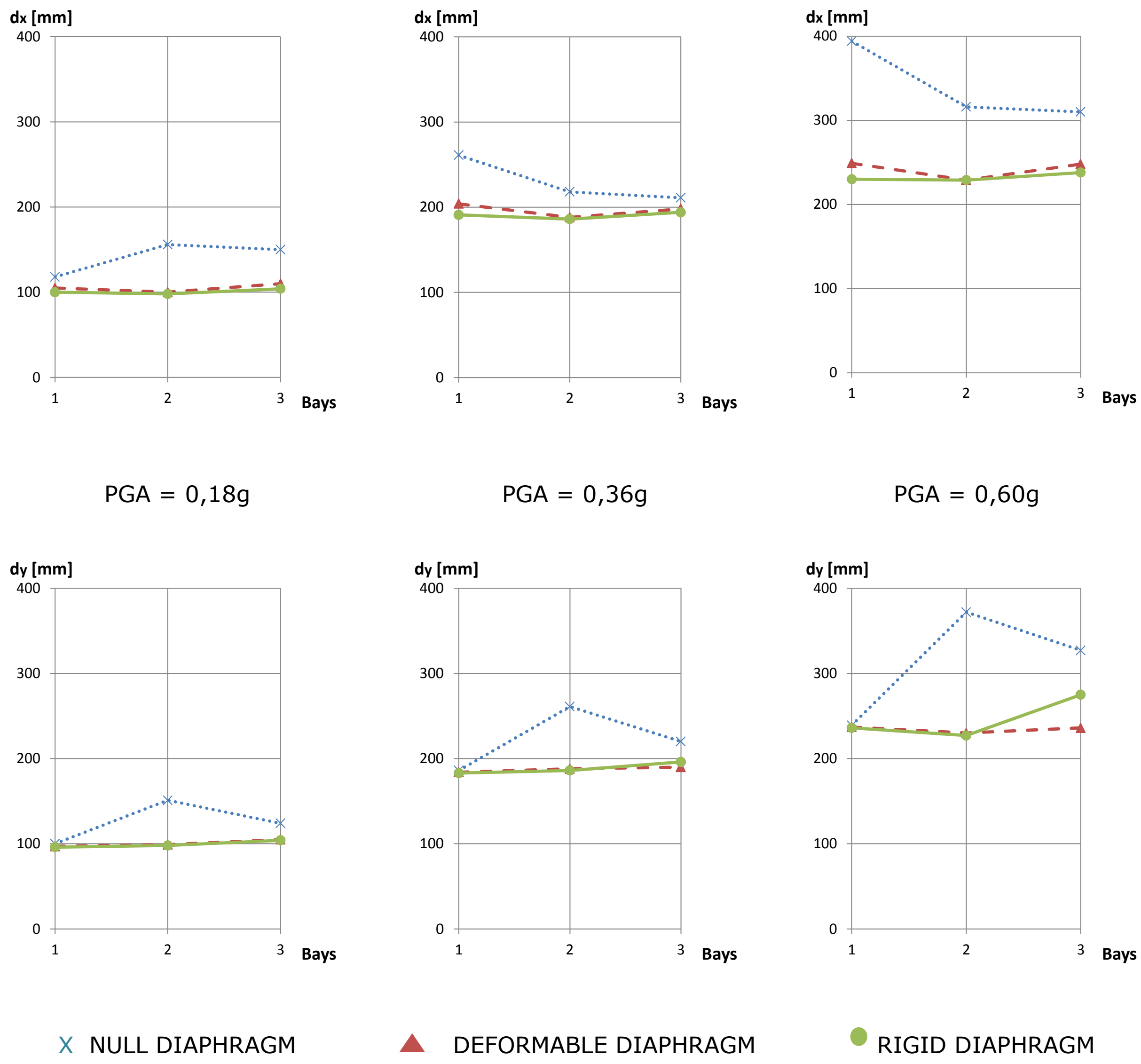
Modes of vibration and periods of vibration for 1 bay structure with vertical panels

2 bay; null diaphragm	**2 bay; deformable diaphragm**	**2 bay; rigid diaphragm**
T1 = 0.88 sec: transverse dir. 	T1 = 0.77 sec: transverse dir. 	T1 =0.76 sec: transverse dir. 
T2 = 0.83 sec: torsional 	T2 = 0.76 sec: longitudinal dir. 	T2 =0.75 sec: torsional 
T3 = 0.81 sec: longitudinal dir. 	T3 = 0.63 sec: torsional 	T3 = 0.62 sec: longitudinal dir. 

Modes of vibration and periods of vibration for 2 bay structure with vertical panels

3 bay; null diaphragm	**3 bay; deformable diaphragm**	**3 bay; rigid diaphragm**
T1 = 0.91 sec: transverse dir. 	T1 = 0.83 sec: transverse dir. 	T1 =0.80 sec: transverse dir. 
T2 = 0.86 sec: torsional 	T2 = 0.81 sec: longitudinal dir. 	T2 =0.80 sec: longitudinal dir. 
T3 = 0.84 sec: longitudinal dir. 	T3 = 0.69 sec: torsional 	T3 = 0.66 sec: torsional 

Modes of vibration and periods of vibration for 3 bay structure with vertical panels

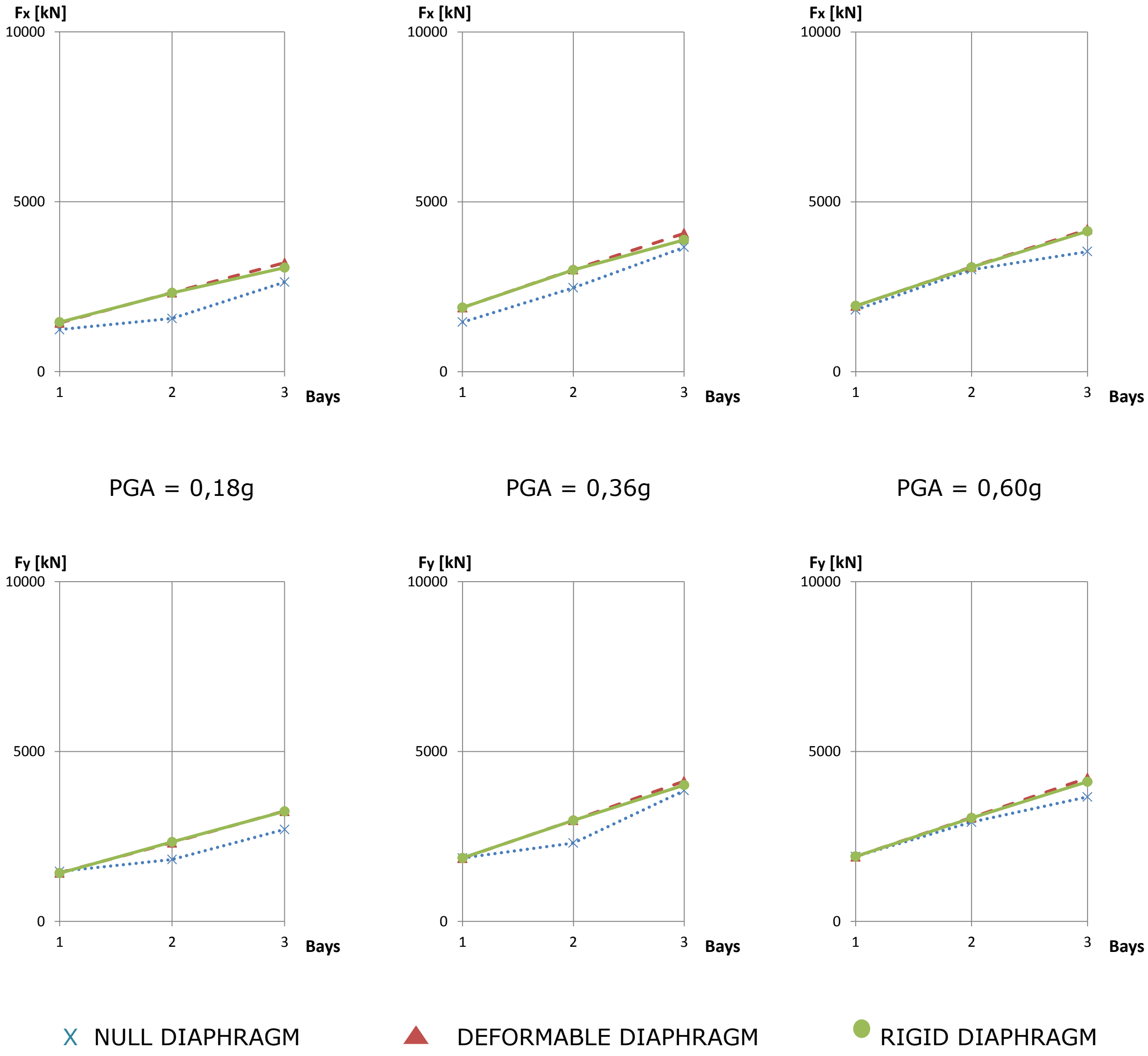


DRIFT (ISOSTATIC)

Comments

For 1-bay the diaphragm action is not relevant in y (longitudinal) direction, is important in x (transverse) direction mainly for the higher levels of excitation.

Deformable and rigid diaphragms have very similar effects in reducing the maximum displacements with respect to the null diaphragm action.



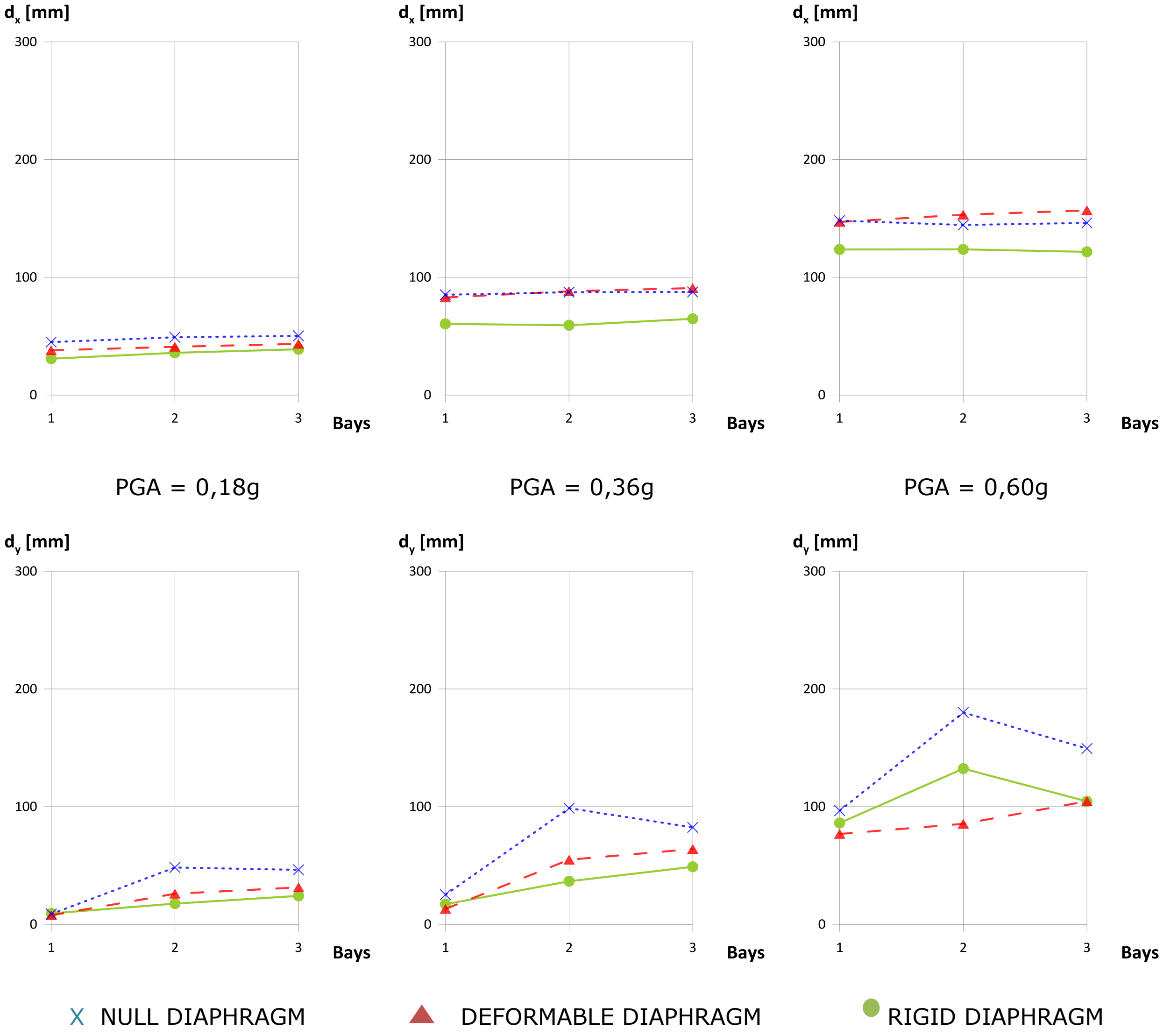
TOTAL BASE SHEAR (ISOSTATIC)

Comments

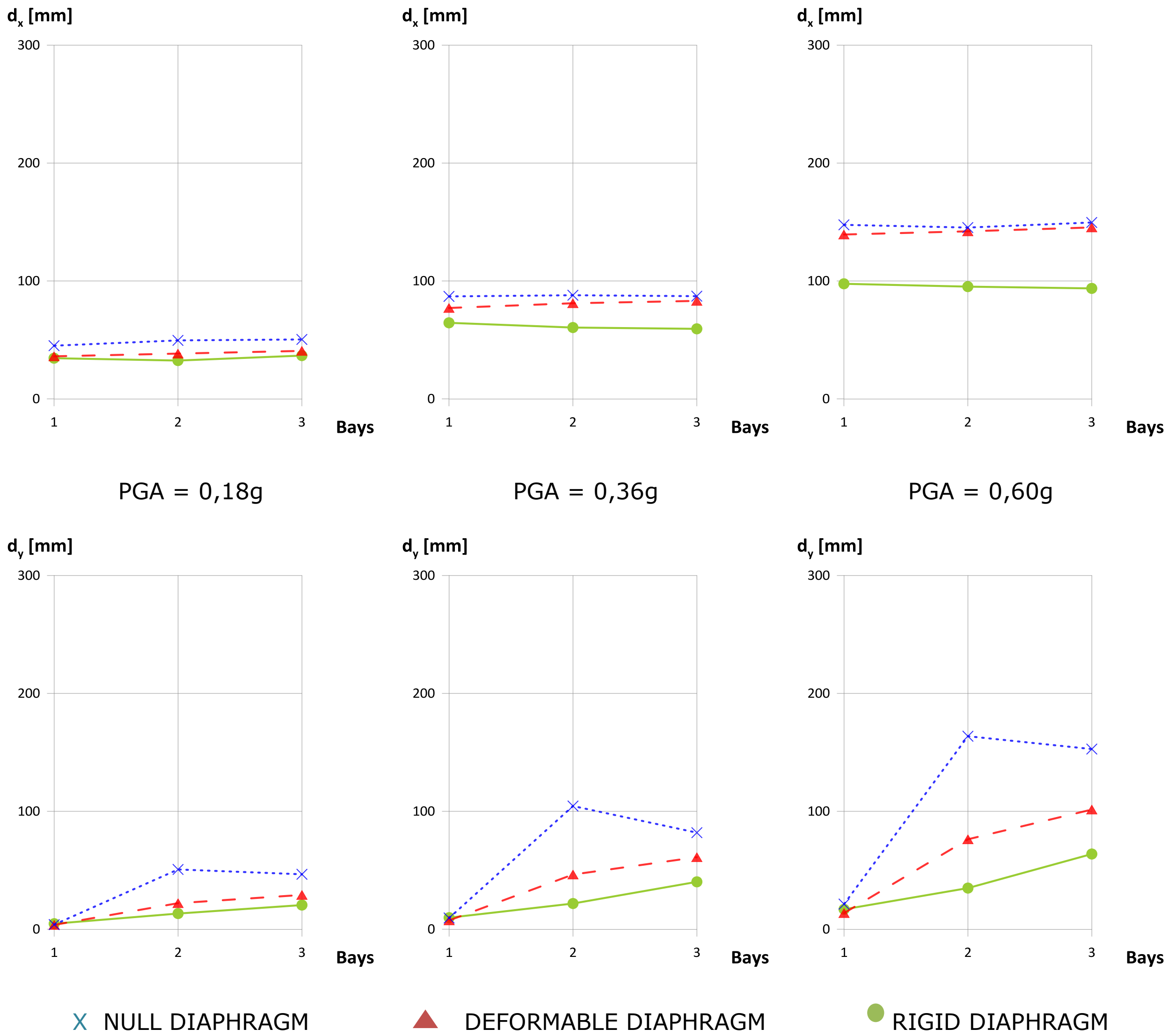
These diagrams represent the ordinary outcome of the present design practice.

There is no relevant influence of the type of roof diaphragm on total base shear.

INTEGRATED SYSTEM OF CONNECTIONS



DRIFT (INTEGRATED PANELS WITH THREE CONNECTIONS)

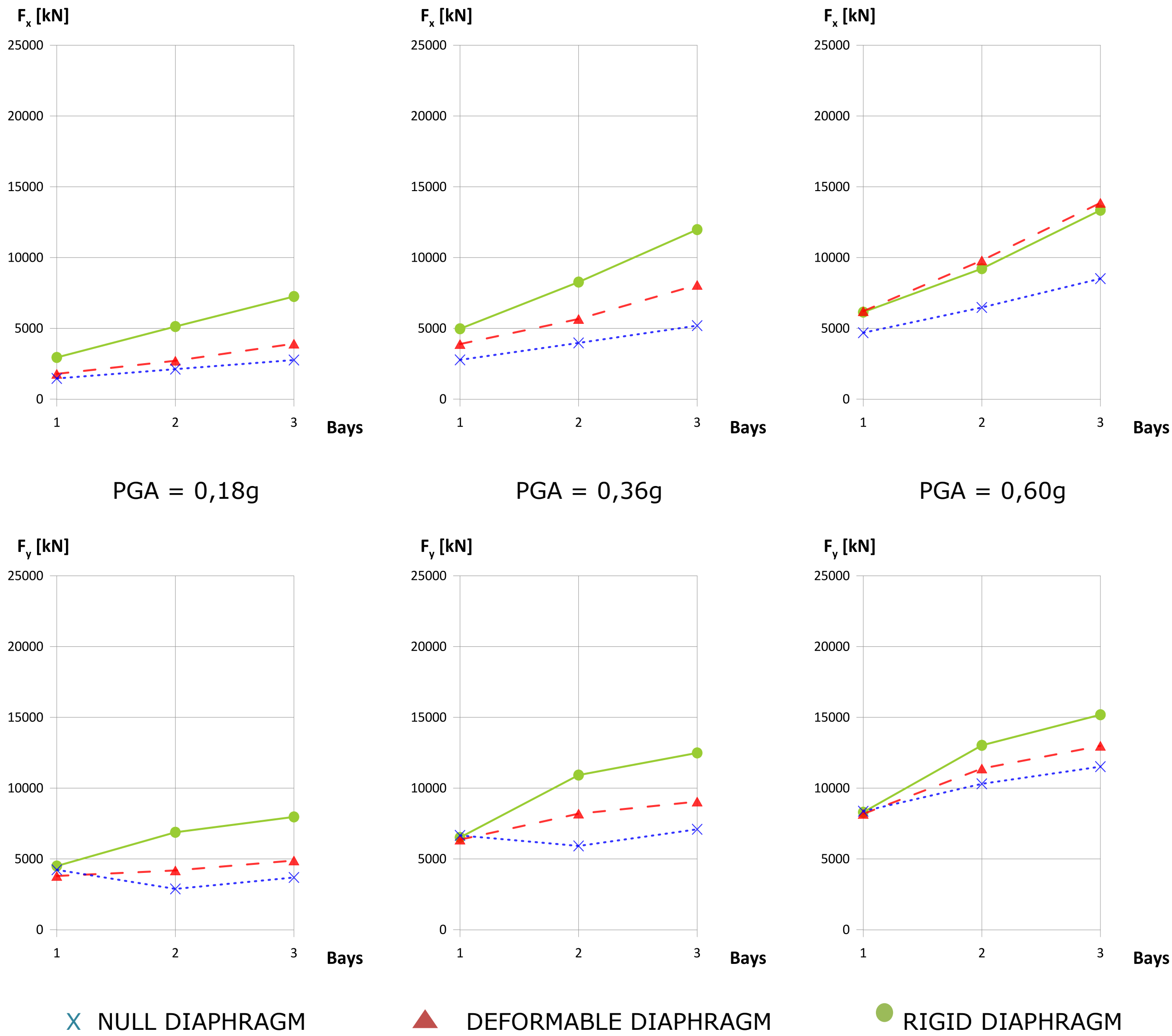


DRIFT (INTEGRATED PANELS WITH FOUR CONNECTIONS)

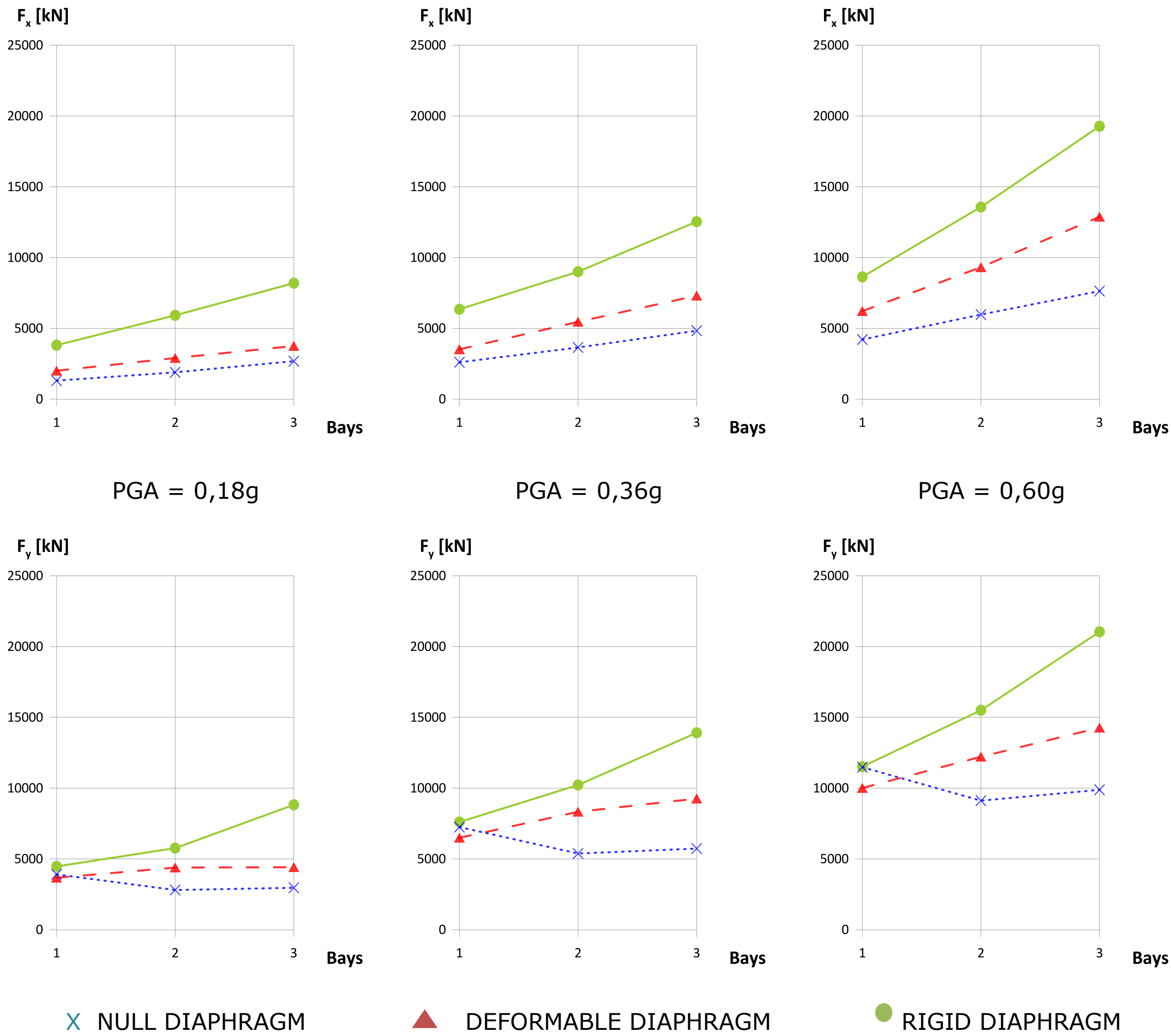
Comments

As expected, null roof diaphragm shows, in general, the largest displacements whereas structures with rigid roof diaphragm show the smallest. However, for 1-bay structure, the displacements along the y (longitudinal) direction are, in most cases, not affected by the roof diaphragm action. For multiple bays, as well as for loading along the x (transverse) direction, the effect of the diaphragm action is significant.

Structures with panels connected at three points experience, in general, displacements of the same magnitude comparing to structures with panels connected at four points.



TOTAL BASE SHEAR (INTEGRATED PANELS WITH THREE CONNECTIONS)

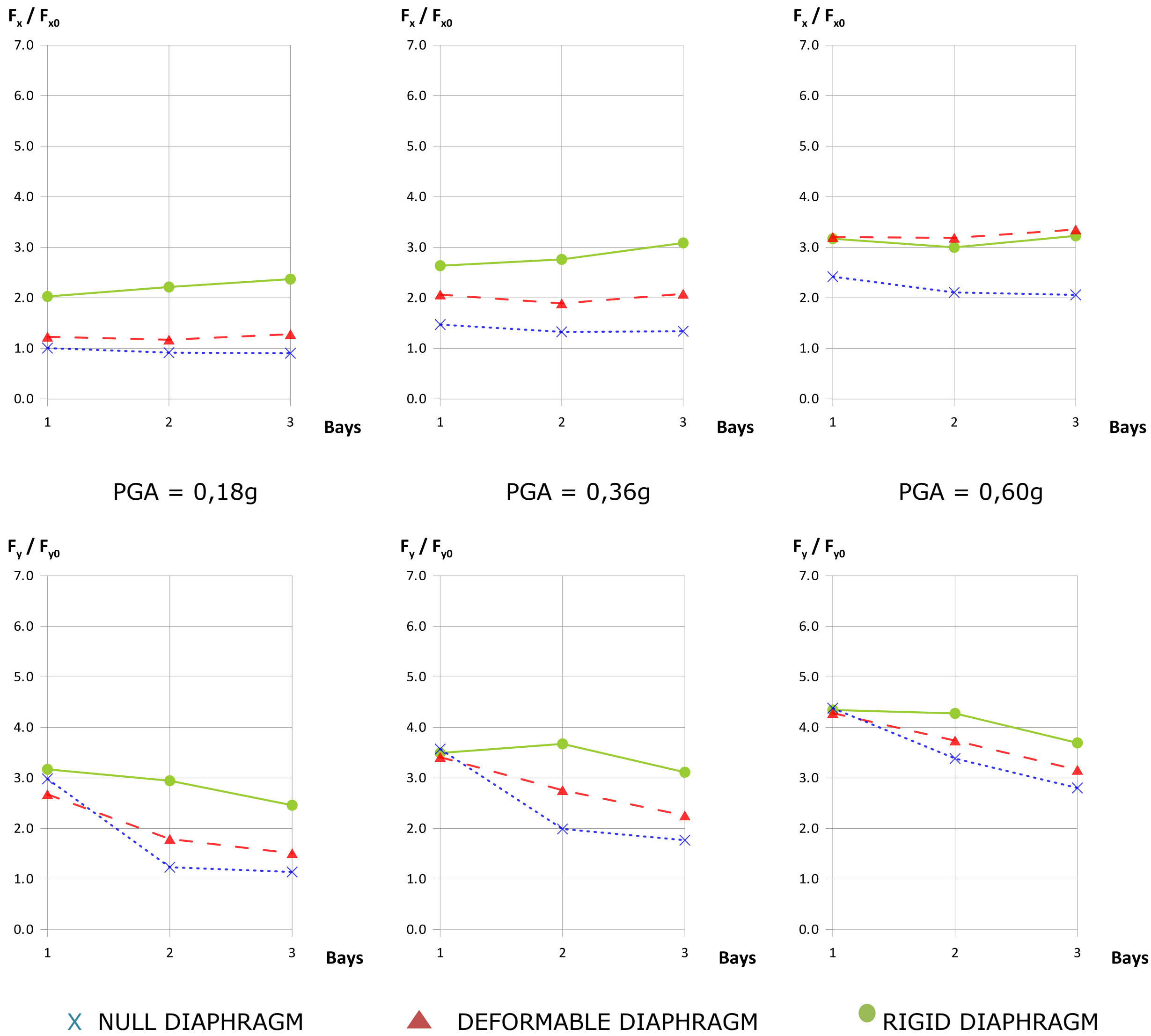


TOTAL BASE SHEAR (INTEGRATED PANELS WITH FOUR CONNECTIONS)

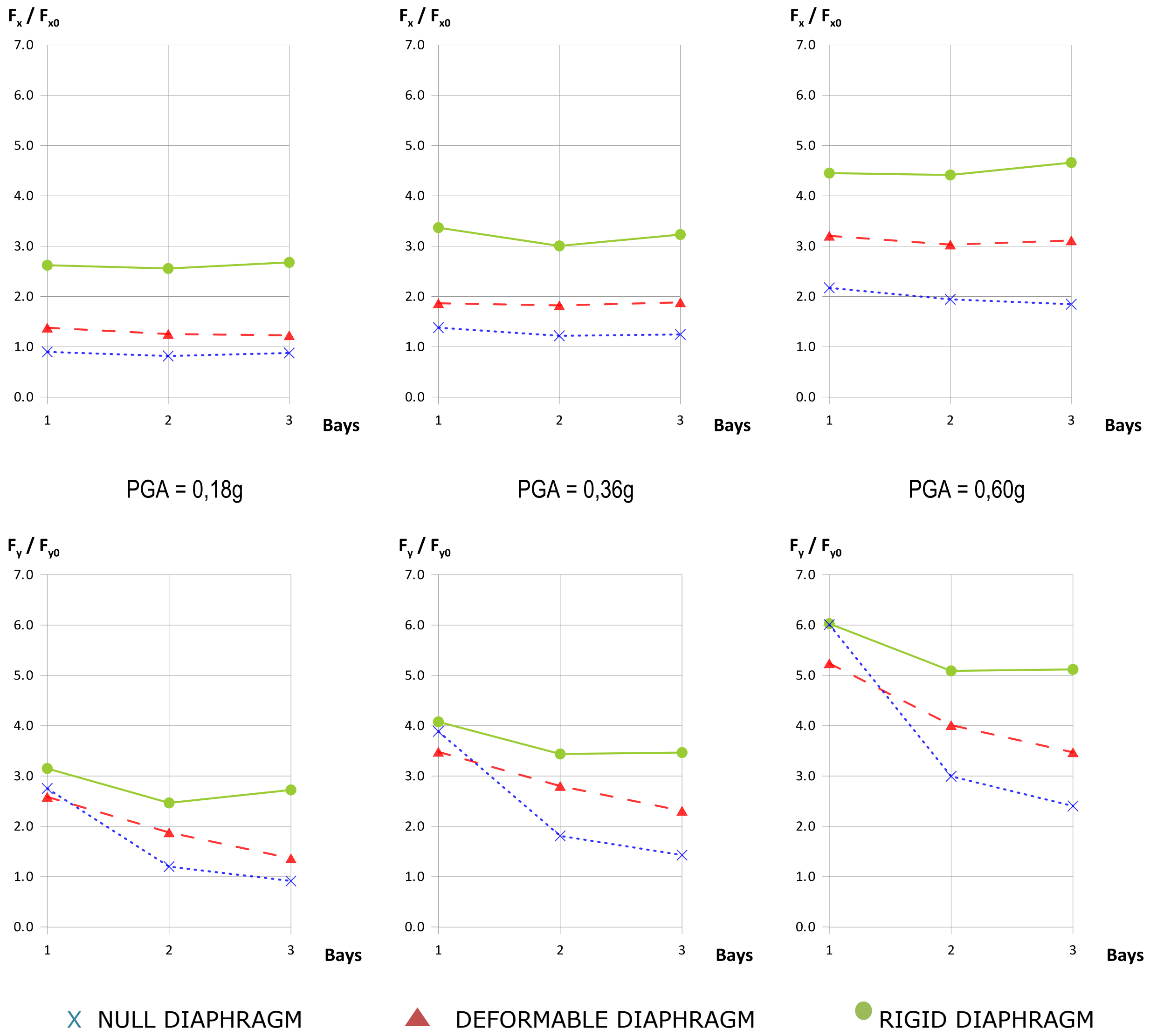
Comments

In integrated systems, the rigidity of the roof affects the base shear along both the x (transverse) and y (longitudinal) direction and it can generally be said that it is larger for structures with stiffer roof configuration. Moreover, the magnitude of the base shear increases with the number of bays, due to the increase in the total mass, as well as with the intensity of the ground shaking.

Structures with panels connected at three points show, in general, lower base shear comparing to structures with panels connected at four points.



TOTAL BASE SHEAR (RELATIVE) (INTEGRATED PANELS WITH THREE CONNECTIONS)

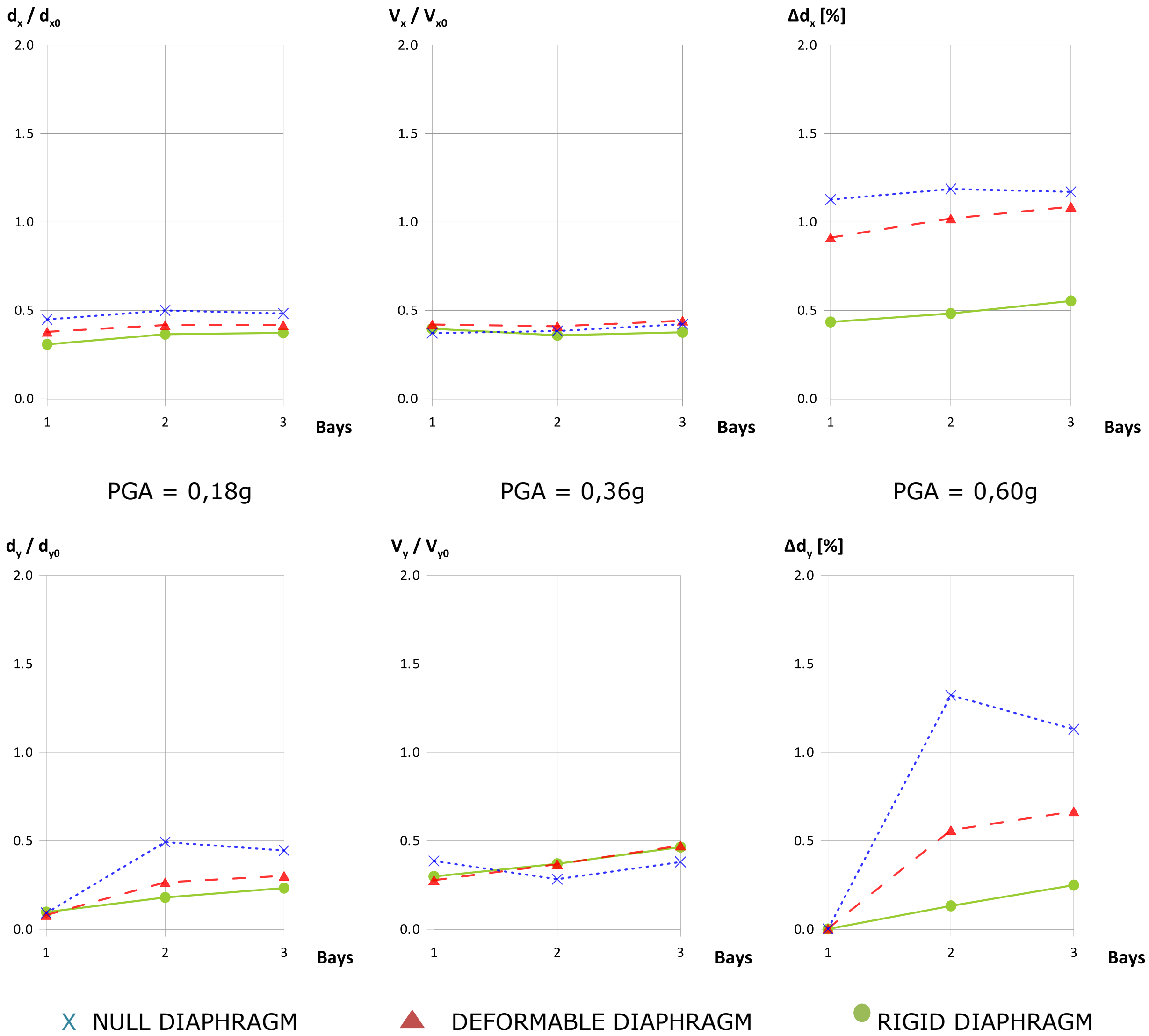


TOTAL BASE SHEAR (RELATIVE) (INTEGRATED PANELS WITH FOUR CONNECTIONS)

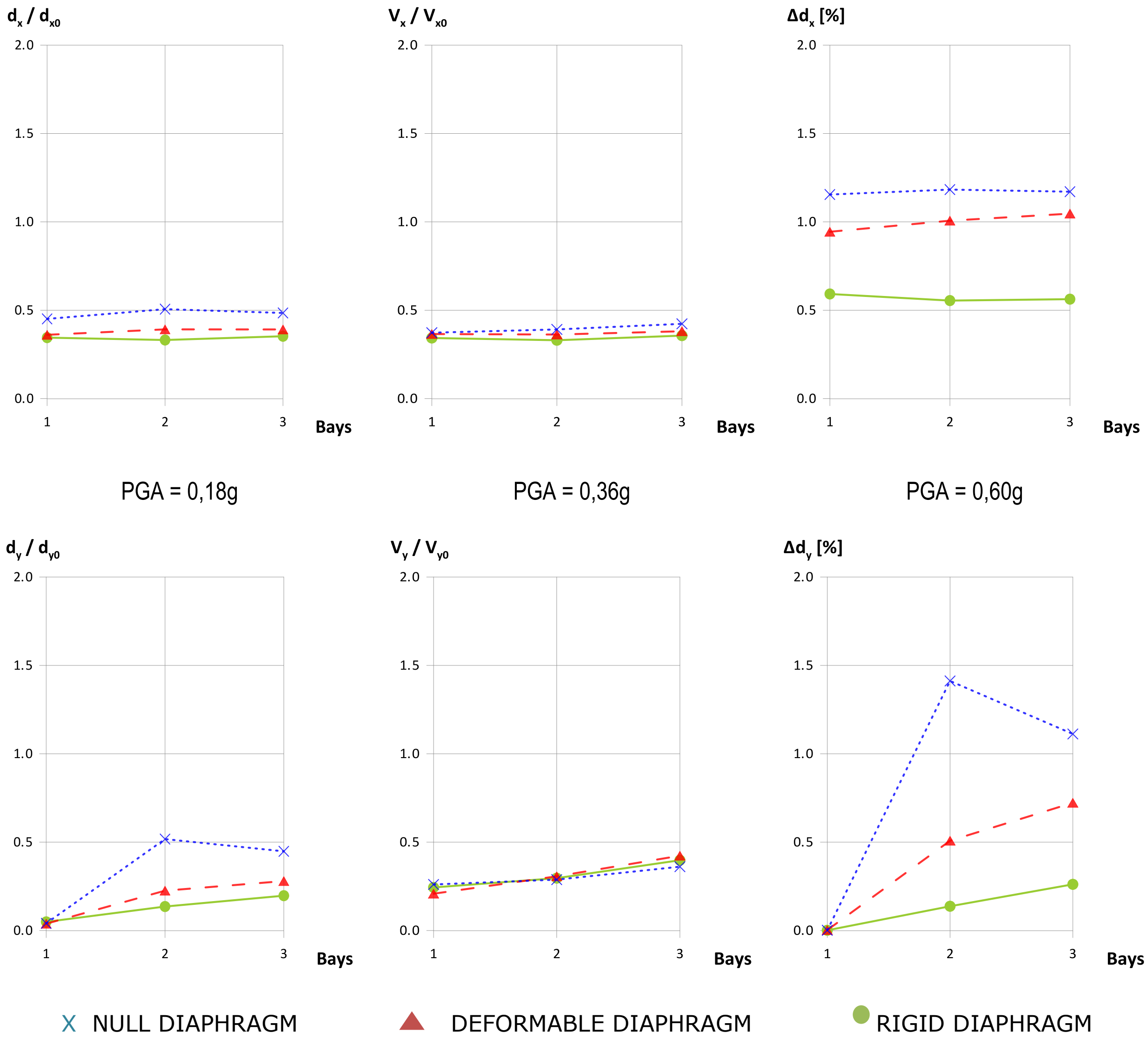
Comments

Comparing to the corresponding isostatic (reference) structures, the base shear forces induced to the integrated systems are generally larger, due to the smaller natural period of the buildings. The relative base shear increases for higher intensity of the ground shaking and stiffer roof diaphragm but the number of bays affect differently the transverse and the longitudinal direction. For loading along the x direction, the ratio is practically constant whereas for loading along the y direction it decreases but still is larger than 1.0.

Structures with panels connected at three points experience, in general, lower relative base shear comparing to structures with panels connected at four points.



COMPLEMENTARY INFORMATION (INTEGRATED PANELS WITH THREE CONNECTIONS)



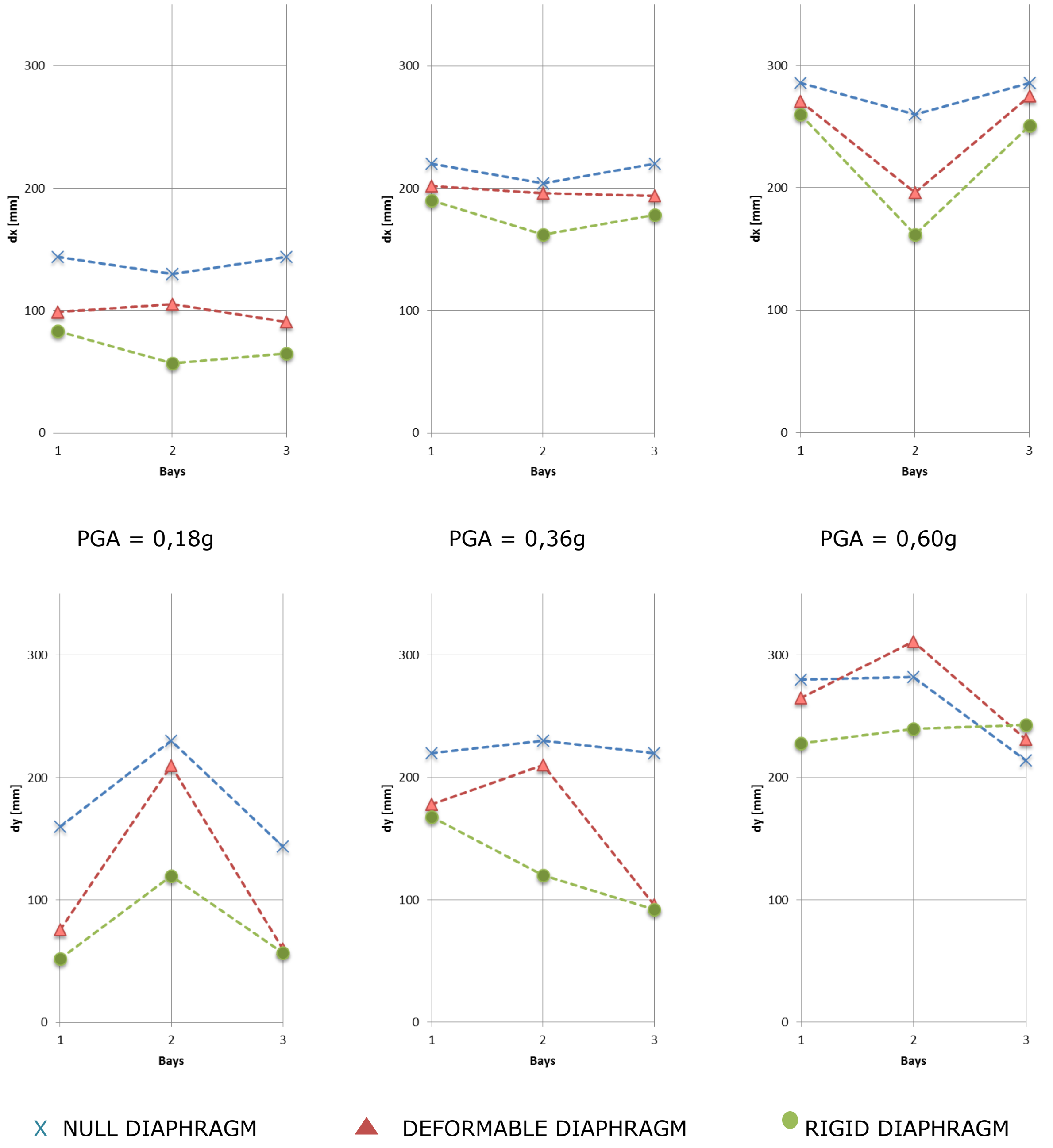
COMPLEMENTARY INFORMATION (INTEGRATED PANELS WITH FOUR CONNECTIONS)

Comments

At the service limit condition (0,18g), the displacements induced to the integrated structures, for all roof configurations, are smaller than the ones of the corresponding isostatic building. The reduction in the displacements is larger for increased roof rigidity.

The stiffening effect of the wall panels lead to a reduction of the shear induced to the columns at the no-collapse limit condition (0,36g), which is generally independent of the diaphragm action of the roof.

PLASTIC DISSIPATIVE SYSTEMS OF CONNECTIONS

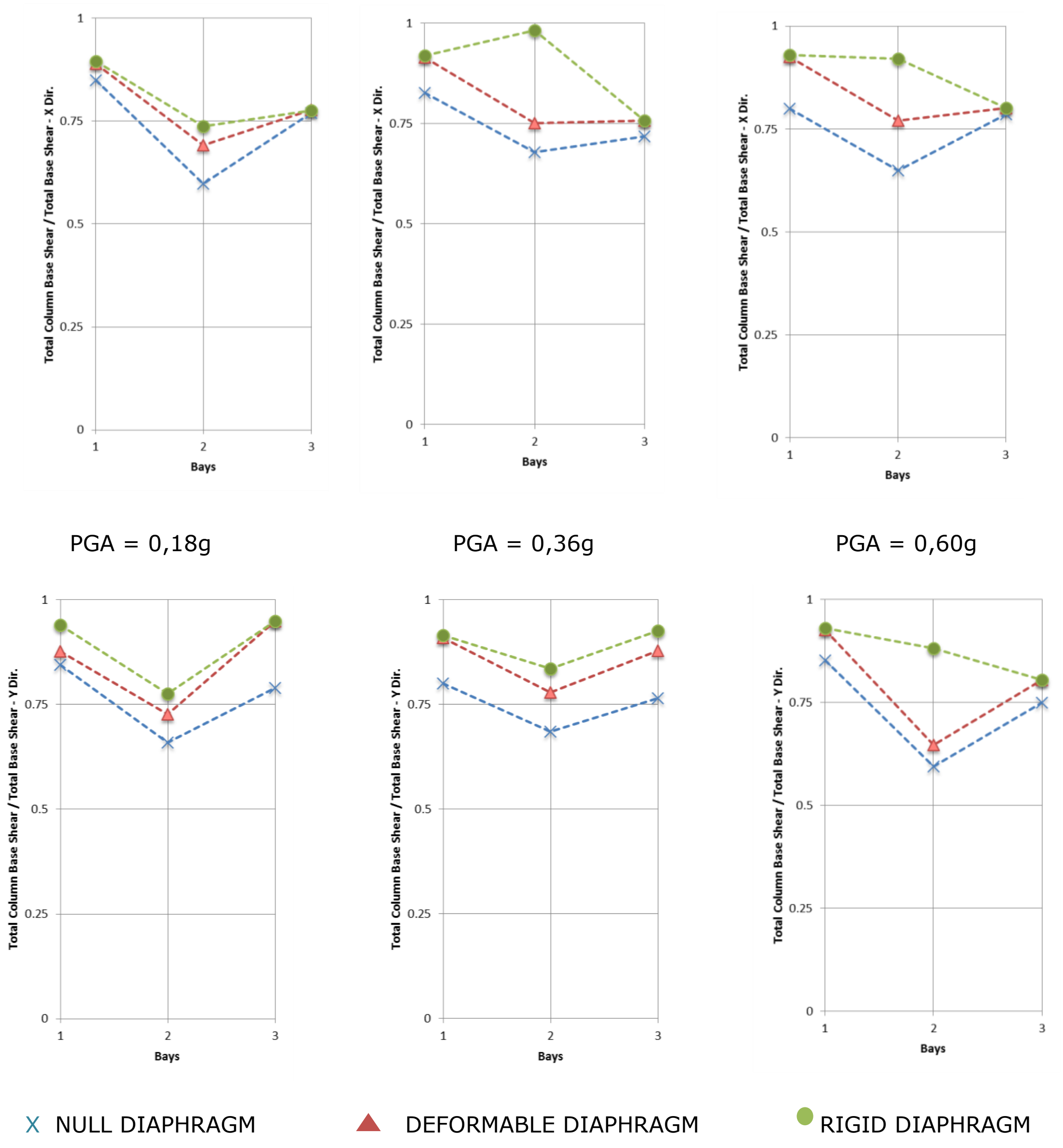


DRIFT (PLASTIC DISSIPATIVE)

Comments

The drifts increase as the diaphragm changes from null to rigid, as a general tendency. This is because the increasing diaphragm action integrates the system and increases the effect of the panels on the overall response, not only on the response of the frame they are attached to.

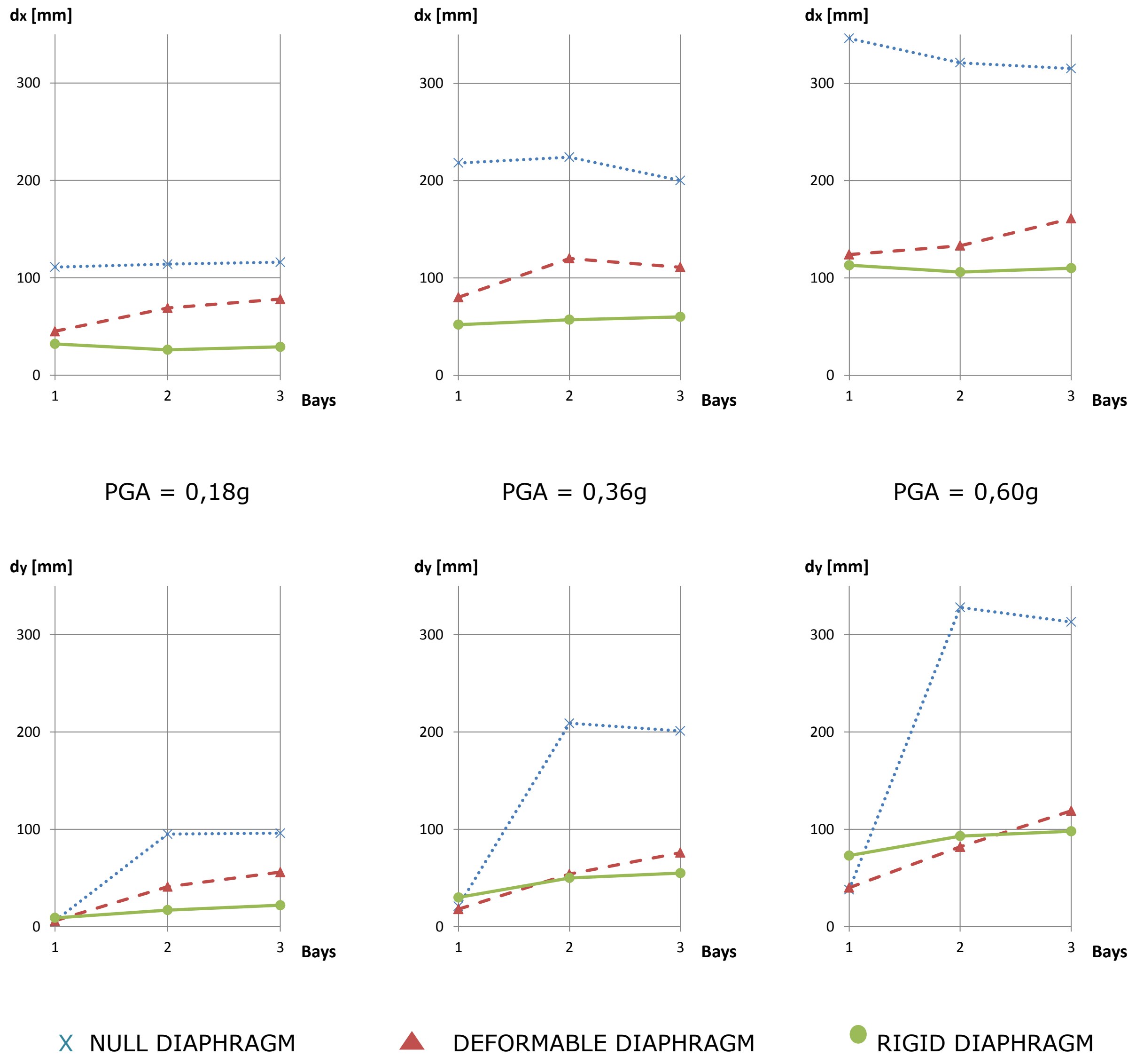
There is not a clear correlation between the number of bays and the overall system drift.



Comments

One of the important aspects of the response in case of plastic dissipative devices is the ratio of the base shear carried by the columns to the total base shear. This ratio is 1.0 in an isostatic system and is very low in an integrated system. The ratio varies between 0 and 1 in case of dissipative systems. The general tendency observed in these analyses, as shown above, is that the contribution of the panels to the overall load bearing increases as value but decreases as ratio.

FRICTION DISSIPATIVE SYSTEMS OF CONNECTIONS



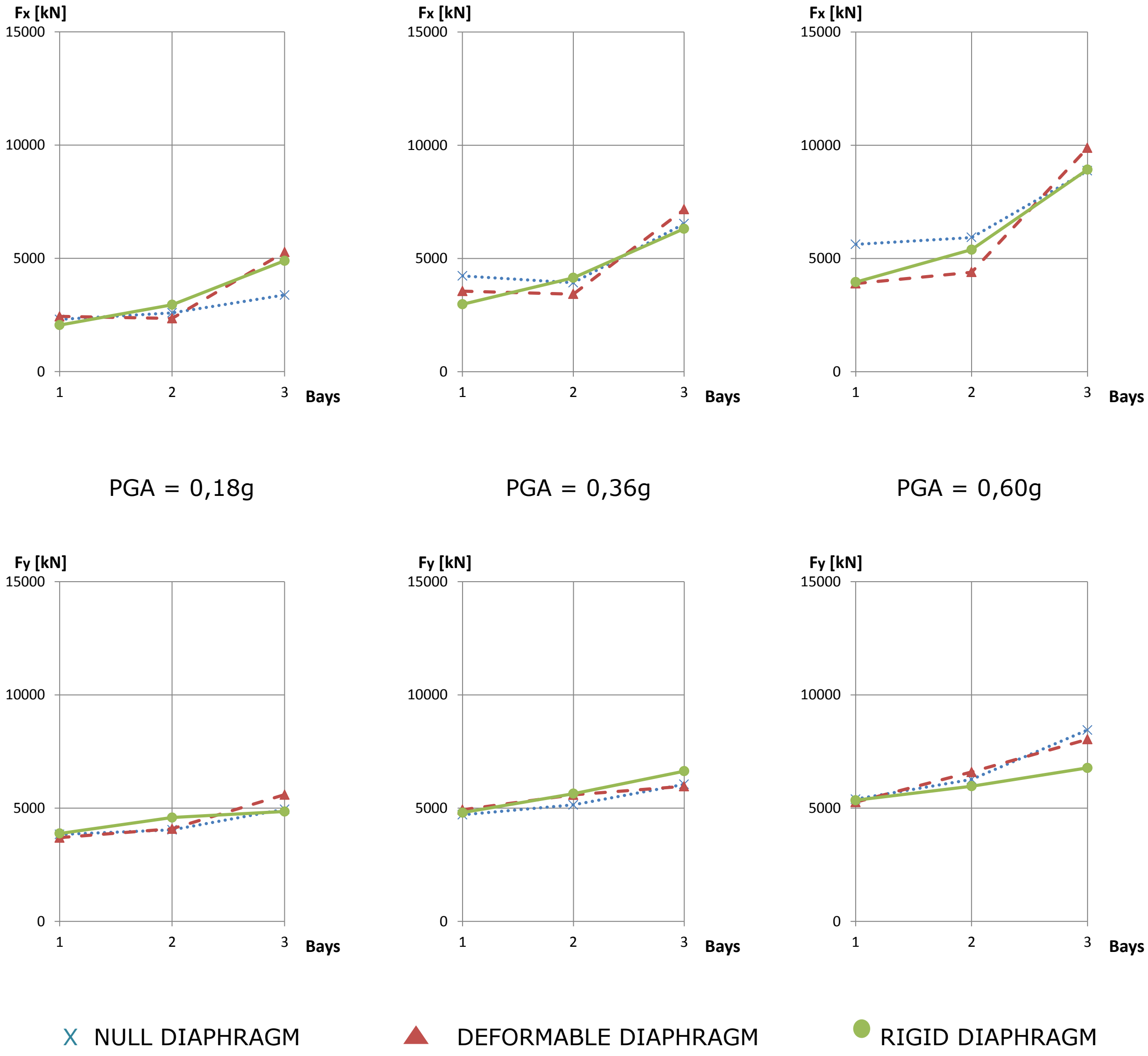
DRIFT (FRICTION DISSIPATIVE)

Comments

For 1-bay the diaphragm action is not relevant in y (longitudinal) direction, is very important in x (transverse) direction.

For 2 and 3-bays the diaphragm action is always very important in reducing the maximum displacements.

Also deformable diaphragms have a relevant influence in reducing the maximum displacements.



TOTAL BASE SHEAR (FRICTION DISSIPATIVE)

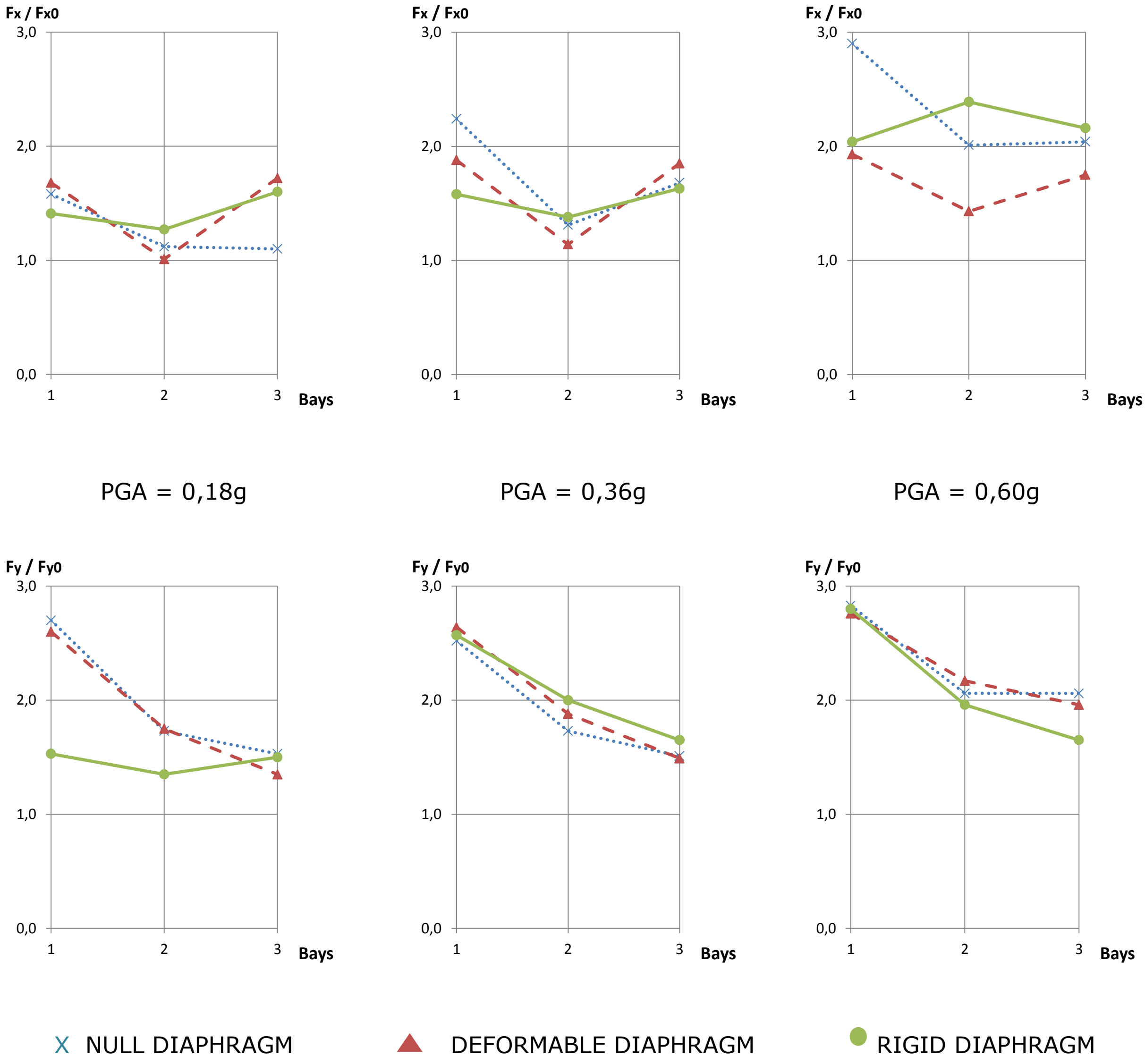
Comments

The influence of the roof diaphragm on the global response of the structure is small.

At service (0,18g) and no-collapse (0,36g) limit conditions for 1 and 2-bays the global response is higher in y (longitudinal) than in x (transverse) direction because of the higher stiffening influence of the wall panels.

Close to collapse (0,60g) conditions the above difference lowers and changes, being affected by the actual distribution of the plastic resources in the structural elements.

In general the global response for increasing number of bays grows less than proportionally to the correspondent involved masses.

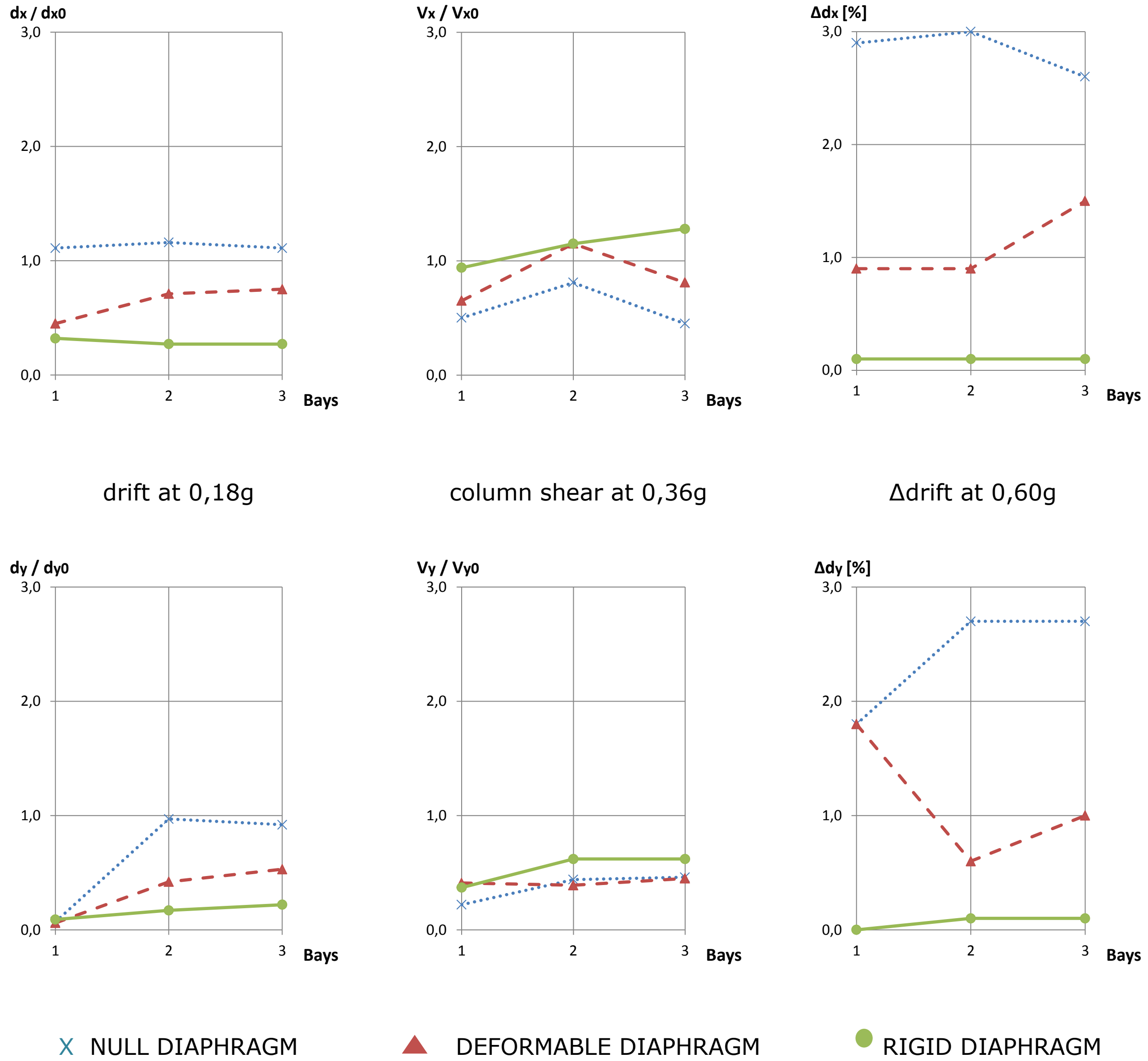


TOTAL BASE SHEAR (RELATIVE) (FRICTION DISSIPATIVE)

Comments

With respect to the isostatic arrangement the stiffening effect of wall panels leads to higher responses that arrive to almost 3 times for 1-bay in y (longitudinal) direction.

In general the above effect decreases with the higher number of bays in y (longitudinal) direction.



COMPLEMENTARY INFORMATION (FRICTION DISSIPATIVE)

Comments

Except for 1-bay in y (longitudinal) direction, at service (0,18g) limit conditions the null diaphragm corresponds to the isostatic arrangement in terms of maximum displacements.

At the service (0,18g) limit condition the deformable and rigid diaphragms lead to a relevant reduction of displacements in both directions with respect to the isostatic arrangement.

The stiffening effect of wall panels, at the no-collapse (0,36g) limit condition, leads to a relevant reduction of the shear in columns in y (longitudinal) direction with respect to the isostatic arrangement.

The above effect is not so relevant in general in x (transverse) direction.

ANNEX A - PROPORTIONING OF TYPE STRUCTURES FOR PARAMETRIC INVESTIGATION

1. Building with long roof elements and short beams: one roof bay

loads:	-	TT70 roof element	3,80 kN/m^2
	-	waterproofing etc.	0,4 kN/m^2
	-	I beam (equivalent $0.4x0.8m^2$ cross section)	8,00 kN/m
	-	cladding panels (panel height about 9m)	
		$4\ kN/m^2$ x 9 m x 2.5 m =	90 kN/panel

Total load effective to earthquake action

(3,8 + 0,4) kN/m^2 x 20 m x 60 m =	5040 kN
8 kN/m x 2 x 60 m =	960 kN
Cladding panels: 90 kN/panel x 64 panels = (accounting for openings)	5760 kN x 0.8
	≅ 4600 kN
half of the cladding panel weight is effective =	2300 kN
Total weight of the vibrating mass W =	8300 kN

Column cross section: 600 mm x 600 mm reinforced with 12 ϕ 20 (ρ_s = 3768/360000 = 1,05%) (see Figure A1)

Column self weight = 0.6 x 0.6 x 7.5 x 25 = 68 kN

N_{Ed} = (3,8+0,4) kN/m^2 x 7.5m x 10 m + 8 kN/m x 7.5 m + 68 kN = 443 kN (façade)

N_{Ed} = (3,8+0,4) kN/m^2 x 3.75m x 10 m + 8 kN/m x 3.75 m + 68 kN = 256 kN (corner)

Column stiffness: I = $(600\ mm)^4/12$ = 108 x 108 mm^4 E = 35 x 103 N/mm^2

$(EI)_{cracked}$ = 0.5 EI = 189 x 1012 Nmm^2 = 189 x 103 kNm^2

h = 7,5 m $3EI/h^3$ = 1344 kN/m

Total stiffness of the building 18 columns $k\delta$ = 18x1344 = 24192 kN/m

Period T = 2 $\sqrt{(W / k\delta)}$ ≅ 1,16 s θ factor for columns = 0,1 (2^{nd} order neglected)

a_g = 0,30g ground type B: S = 1,2 q = 3,0

S_d = 0,30g x1,2 x (2,5 / 3,0) x 0,50 / 1,16 = 0,1293g

Maximum static force equivalent to the earthquake

E_d = 0,1293 x 8300 = 1073 kN

M_{Ed} = 1073 x 7,5 / 18 = 447 kNm

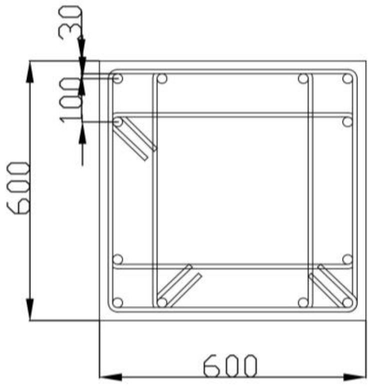
M_{Rd} = 487 kNm (corner columns)

M_{Rd} = 533 kNm (facade columns)

Drift at SLS $d = 1073 \times 0,5 \times 3,0 / 24192 = 0,0665$ m (= 0,887 %)

Maximum value of static force equivalent to the earthquake withstood by the structure:

$E_{max} = (4 \times 487 + 14 \times 533) / 7,5 = 1255$ kN



STEEL CLASS B450C

CONCRETE CLASS C45/55

LONGITUDINAL BARS 12Φ20

STIRRUPS Φ10

CLEAR COVER 30 mm

Figure A1

2. building with long roof elements and short beams: two roof bays

loads:	-	TT70 roof element	3,80 kN/m^2
	-	waterproofing etc.	0,4 kN/m^2
	-	I beam (equivalent 0.4x0.8m^2 cross section)	8,00 kN/m
	-	cladding panels (panel height about 9m)	
		4 kN/m2 x 9 m x 2,5 m =	90 kN/panel

Total load effective to earthquake action

(3.8 + 0.4) kN/m^2 x 40 m x 60 m =	10080 kN
8 kN/m x 3 x 60 m =	1440 kN
Cladding panels: 90 kN/panel x 80 panels = (accounting for openings)	7200 kN x 0.8
	≅ 5760 kN
half of the cladding panel weight is effective =	2880 kN
Total weight of the vibrating mass W =	14320 kN

Column cross section: 600 mm x 600 mm reinforced with 12 ϕ 22 (ρ_s = 4560/360000 = 1,27%) (see Figure A2)

Column self weight = 0.6 x 0.6 x 7.5 x 25 = 68 kN

N_{Ed} = (3,8+0,4) kN/m$_2$ x 7.5m x 20 m + 8 kN/m x 7.5 m + 68 kN = 758 kN (central)

N_{Ed} = (3,8+0,4) kN/m$_2$ x 7.5m x 10 m + 8 kN/m x 7.5 m + 68 kN = 443 kN (façade)

N_{Ed} = (3,8+0,4) kN/m$_2$ x 3.75m x 10 m + 8 kN/m x 3.75 m + 68 kN = 256 kN (corner)

Column stiffness: I = (600 mm)^4/12 = 108 x 108 mm^4 E = 35 x 103 N/mm^2

$(EI)_{cracked}$ = 0,5 EI = 189 x 1012 Nmm^2 = 189 x 103 kNm^2

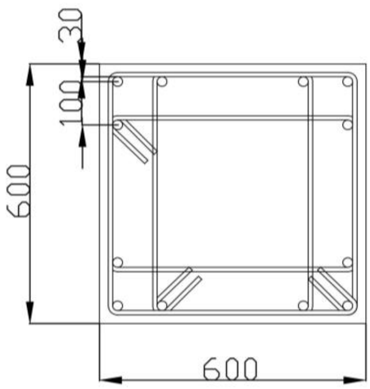
h = 7.5 m 3EI/h^3 = 1344 kN/m

Total stiffness of the building 27 columns kδ = 36288 kN/m

Period T = 2 $\sqrt{(W/k\delta)}$ ≅ 1,24 sec θ factor = 0,133 (2nd order factor 1,15)

a_g = 0,30g ground type B: S = 1,2 q = 3,0

S_d = 0,30g x1,2 x (2,5 / 3,0) x 0,50 / 1,24 = 0,1210g



STEEL CLASS B450C

CONCRETE CLASS C45/55

LONGITUDINAL BARS 12Φ22

STIRRUPS Φ10

CLEAR COVER 30 mm

Figure A2

Maximum static force equivalent to the earthquake

E_d = 0,1210 x 14320 = 1733 kN

M_{Ed} = 1,15 x 1733 x 7,5 / 27 = 554 kNm

M_{Rd} = 569 kNm (corner columns)

M_{Rd} = 614 kNm (facade columns)

M_{Rd} = 690 kNm (central)

Drift at SLS d = 1733 x 0,5 x 3,0 / 36288 = 0,0716 m (= 0,955 %)

Maximum value of static force equivalent to the earthquake withstood by the structure:

E_{max} = (4 x 569 + 16 x 614 + 7 x 690) / (1,15 x 7,5) = 1963 kN

3. building with long roof elements and short beams: three roof bays

loads: - TT70 roof element 3,80 kN/m^2

- waterproofing etc. 0,4 kN/m^2
- I beam (equivalent 0.4x0.8m^2 cross section) 8,00 kN/m
- cladding panels (panel height about 9m)

4 kN/m2 x 9 m x 2.5 m = 90 kN/panel

Total load effective to earthquake action

(3,8 + 0,4) kN/m2 x 60 m x 60 m =	15120 kN
8 kN/m x 4 x 60 m =	1920 kN
Cladding panels: 90 kN/panel x 96 panels = (accounting for openings)	8640 kN x 0.8
	≅ 6900 kN
half of the cladding panel weight is effective =	3450 kN
Total weight of the vibrating mass W =	20490 kN

Column cross section: 600 mm x 600 mm reinforced with 12 ϕ 24 (ρ_s = 5888/360000 = 1.64%) (see Figure A3)

Column self weight = 0.6 x 0.6 x 7.5 x 25 = 68 kN

N_{Ed} = (3,8+0,4) kN/m^2 x 7,5m x 20 m + 8 kN/m x 7,5 m + 68 kN = 758 kN (central)

N_{Ed} = (3,8+0,4) kN/m^2 x 7,5m x 10 m + 8 kN/m x 7,5 m + 68 kN = 443 kN (lateral)

N_{Ed} = (3,8+0,4) kN/m^2 x 3,75m x 20 m + 8 kN/m x 3,75 m + 68 kN = 345 kN (front)

N_{Ed} = (3,8+0,4) kN/m^2 x 3,75m x 10 m + 8 kN/m x 3,75 m + 68 kN = 256 kN (corner)

Column stiffness: I = (600 mm)^4/12 = 108 x 108 mm^4 - E = 35 x 103 N/mm^2

$(EI)_{cracked}$ = 0.5 EI = 189 x 1012 Nmm^2 = 189 x 103 kNm^2

h = 7,5 m - 3EI/h^3 = 1344 kN/m

Total stiffness of the building 36 columns $k\delta$ = 48384 kN/m

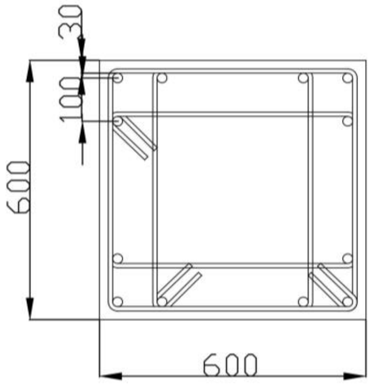
Period T = 2 $\sqrt{}$(W/kδ) ≅ 1,30 sec θ factor = 0,127 (2nd order factor 1,15)

a_g = 0,30g ground type B: S = 1,2 q = 3,0

S_d = 0,30g x1,2 x (2,5 / 3,0) x 0,50 / 1,30 = 0,1154g

Maximum static force equivalent to the earthquake

E_d = 0,1154 x 20490 = 2365 kN



STEEL CLASS B450C

CONCRETE CLASS C45/55

LONGITUDINAL BARS 12Φ24

STIRRUPS Φ10

CLEAR COVER 30 mm

Figure A3

M_{Ed} = 1,15 x 2365 x 7,5 / 36 = 567 kNm

M_{Rd} = 657 kNm (corner columns)

M_{Rd} = 702 kNm (facade columns)

M_{Rd} = 777 kNm (central columns)

Drift at SLS d = 2365 x 0,5 x 3,0 / 48384 = 0,0733 m (= 0,978 %)

Maximum value of static force equivalent to the earthquake withstood by the structure:

E_{max} = (4 x 657 + 18 x 702 + 14 x 777) / (1,15 x 7,5) = 3031 kN

ANNEX B - ANALYSES OF MULTI-STOREY BUILDINGS WITH INTEGRATED CONNECTIONS

The following pages contain the 6 tables of results of the analyses performed on the three-storey buildings with vertical cladding panels attached to the structure with integrated systems of connections.

Two arrangements of panel connections were examined in the analyses: (a) panels with four connections (two with the bottom beam and two with the top beam); and (b) panels with three connections (two with the bottom beam and one with the top beam).

In the following tables the y- and x-direction of loading correspond respectively to the direction of the central beam and to orthogonal one of the plan of Figure B1.

The considered three-storey frame building is similar to the one tested in full scale the SAFECAST project and it is representative of a real three-storey building with two 7.0 m bays in each horizontal direction. The building height is 9.9 m above the foundation, with floor heights equal to 3.5 m, 3.2 m and 3.2 m for the 1st, 2nd and 3rd storey respectively.

In reference to the SAFECAST project, the examined building has the following different characteristics:

- vertical and horizontal panels
- hinged beam-to-column connections
- same floor configuration in all storeys

In each storey totally 12 vertical panels (6 in each direction) with dimensions 0.20 m x 1.80 m are placed along the perimeter, while horizontal panels were used to connect these vertical panels at the floor levels. Vertical panels of 12.0 m height were initially planned to be used, covering the whole height of the building; however, due to the specific features of the integrated connections, it was decided to divide them in three parts, so that individual panels were placed at each floor. Additionally, in the analyses, the horizontal panels were considered to contribute only as masses.

A typical floor plan and side view of the building are shown in Figure B1 and Figure B2 respectively. Due to the large forces developed in panel-to-beam connections, the location of the panels had to be slightly changed in order to be properly connected to the beams, as shown in Figure B1. Moreover, stronger beams than the originally suggested were considered for the perimeter of the building. Regarding the floor configuration, TT-roof elements were used in all storeys with an uniform width.

For the multi-storey building, the following parameters are investigated in parametric analyses:

- Rigid roof diaphragm
- Non-yielding roof-to-beam connections
- Vertical cladding walls with 3 and 4 connections each

Excitation: Tolmezzo (1976) in x- and y-direction for three intensities: 0.18 g, 0.36 g and 0.60 g

For the design the following assumptions are made:

- Steel class: B450C
- Concrete grade: C45/55

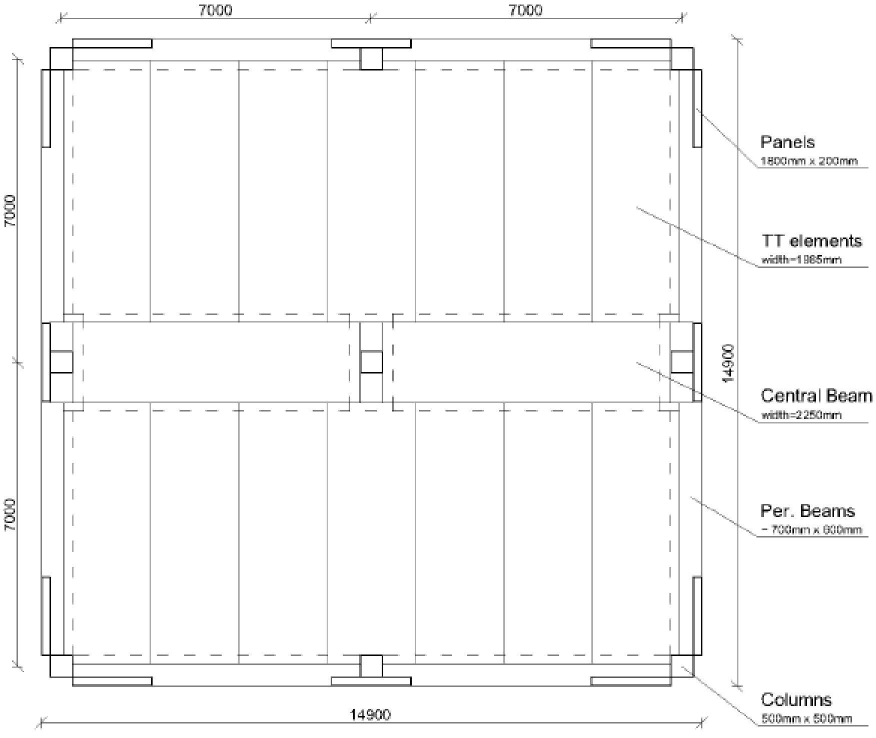


Figure B1

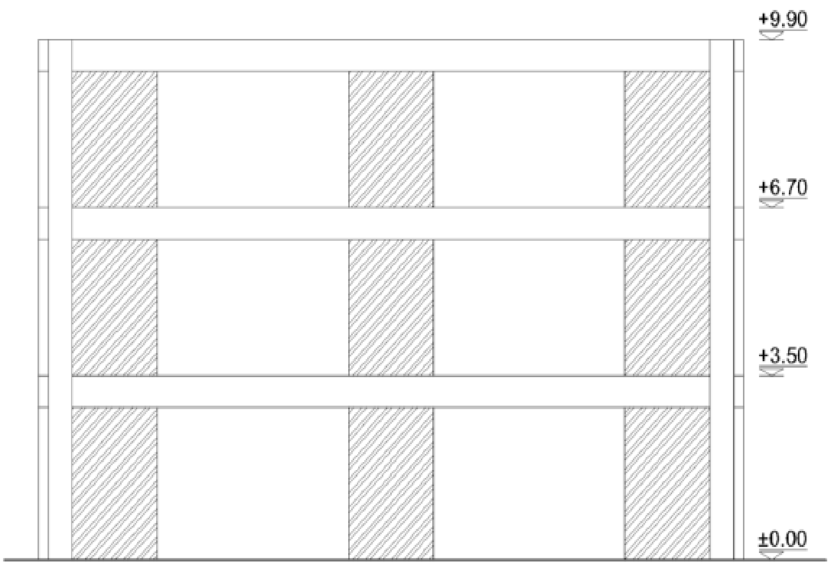


Figure B2

Columns: Rectangular cross section 500 mm x 500 mm (Figure B3)

- Longitudinal reinforcement: 8ϕ20
- Stirrups ϕ8/75 mm

Central Beams: Hollow cross section with dimensions 2250 mm x 40 mm (Figure B4).

The following members had to be modified comparing to the original SAFECAST structure:

- Perimeter Beams: Cross section with dimensions 700mm x 600 mm (Figure B5).
- Panels: Rectangular cross section 1800 mm x 200 mm.

- Floor: TT-elements with cross section as shown in Figure B6.

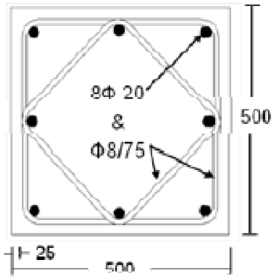


Figure B3

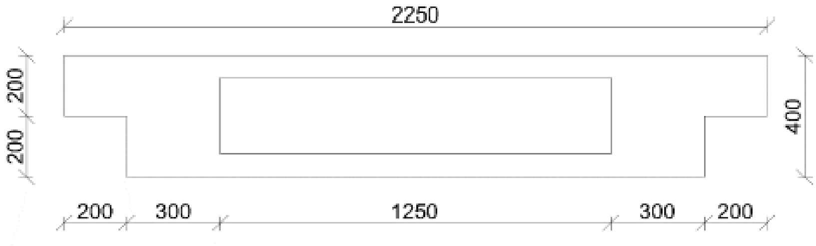


Figure B4

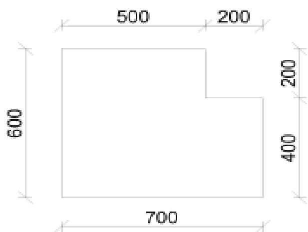


Figure B5

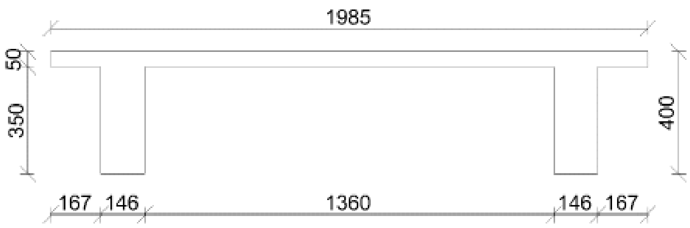


Figure B6

Regarding the masses present on the structure, the following assumptions were made:

- Dead loads:

• Columns	6.25 kN/m
• Central Beams	9.88 kN/m
• Perimeter Beams	9.50 kN/m
• TT-elements	5.04 kN/m
• Panels	9.00 kN/m

- Live loads:
  - Perimeter Beams (due to horizontal panels):

    1^{st} and 2^{nd} storey (Total) 35.44 kN

    3^{rd} storey (Total) 54.25 kN
  - TT-elements:

    1^{st} and 2^{nd} storey (Total) 15.26 kN

    3^{rd} storey (Total) 2.78 kN

It should be noted that, due to the difference in real and model dimensions of the various elements, the aforementioned masses had to be adjusted, so that the masses used in each element of the model had the total element mass presented above.

Response of the 1st floor - panels with four-connections.

INT3/1st	y-direction			x-direction		
Quantity	0.18 g	0.36 g	0.60 g	0.18 g	0.36 g	0.60 g
Maximum drift [mm]	5	13	23	6	13	24
Maximum relative drift [mm]	5	13	23	6	13	24
Ratio [%]	0.2	0.4	0.7	0.2	0.4	0.7
Differential top drift [mm]	1	1	1	0	1	1
Maximum top drift [mm]	5	13	23	6	13	24
Max force roof-roof [kN]	3	5	5	3	5	6
Relative ( )	-	-	-	-	-	-
Max horiz. force roof-beam [kN]	87	105	122	78	88	99
Relative ( )	-	-	-	-	-	-
Max horiz. force beam-column [kN]	103	118	134	80	92	140
Relative ( )	-	-	-	-	-	-
Max force panel-beam [kN]	121	215	324	124	216	322
Relative ( )		-	-	-	-	-
Total floor shear [kN]	1853	3232	4631	1877	3255	4594
Relative ( )	-	-	-	-	-	-
Total column shear [kN]	808	1309	1616	827	1322	1613
Relative ( )	0.44	0.40	0.35	0.44	0.41	0.35
Mean column shear [kN]	90	145	180	92	147	179
Relative ( )	-	-	-	-	-	-
Max column shear [kN]	98	153	184	101	157	190
Relative ( )	1.09	1.05	1.03	1.10	1.07	1.06

Table B1

Response of the 2nd floor - panels with four-connections.

INT3/2nd	y-direction			x-direction		
Quantity	0.18 g	0.36 g	0.60 g	0.18 g	0.36 g	0.60 g
Maximum drift [mm]	14	32	58	14	33	59
Maximum relative drift [mm]	8	20	35	9	20	35
Ratio [%]	0.3	0.6	1.1	0.3	0.6	1.1
Differential top drift [mm]	1	1	1	1	1	2
Ratio [%]	0.0	0.0	0.0	0.0	0.0	0.1
Max force roof-roof [kN]	4	6	10	5	7	11
Relative ( )	-	-	-	-	-	-
Max horiz. force roof-beam [kN]	95	115	150	94	115	146
Relative ( )	-	-	-	-	-	-
Max horiz. force beam-column [kN]	98	118	126	105	157	229
Relative ( )	-	-	-	-	-	-
Max force panel-beam [kN]	142	237	359	143	240	361
Relative ( )	-	-	-	-	-	-
Total floor shear [kN]	1490	2695	3753	1520	2716	3748
Relative ( )	-	-	-	-	-	-
Total column shear [kN]	277	486	629	266	480	598
Relative ( )	0.19	0.18	0.17	0.18	0.18	0.16
Mean column shear [kN]	31	54	70	30	53	67
Relative ( )	-	-	-	-	-	-
Max column shear [kN]	49	77	98	48	76	86
Relative ( )	1.60	1.43	1.39	1.63	1.42	1.29

Table B2

Response of the 3rd floor - panels with four-connections.

INT3/3rd	y-direction			x-direction		
Quantity	0.18 g	0.36 g	0.60 g	0.18 g	0.36 g	0.60 g
Maximum drift [mm]	21	47	84	22	48	86
Maximum relative drift [mm]	8	15	26	8	16	27
Ratio [%]	0.3	0.5	0.8	0.3	0.5	0.9
Differential top drift [mm]	1	2	3	2	3	4
Ratio [%]	0.0	0.1	0.1	0.1	0.1	0.1
Max force roof-roof [kN]	10	16	22	10	16	24
Relative ( )	-	-	-	-	-	-
Max horiz. force roof-beam [kN]	98	133	177	113	159	226
Relative ( )	-	-	-	-	-	-
Max horiz. force beam-column [kN]	94	156	212	163	247	377
Relative ( )	-	-	-	-	-	-
Max force panel-beam [kN]	128	194	285	131	199	292
Relative ( )	-	-	-	-	-	-
Total floor shear [kN]	1035	1558	2369	1020	1535	2364
Relative ( )	-	-	-	-	-	-
Total column shear [kN]	259	440	550	268	476	598
Relative ( )	0.25	0.28	0.23	0.26	0.31	0.25
Mean column shear [kN]	29	49	61	30	53	66
Relative ( )	-	-	-	-	-	-
Max column shear [kN]	35	57	70	36	66	82
Relative ( )	1.20	1.16	1.14	1.22	1.25	1.24

Table B3

Response of the 1st floor - panels with three-connections.

INT3/1st	y-direction			x-direction		
Quantity	0.18 g	0.36 g	0.60 g	0.18 g	0.36 g	0.60 g
Maximum drift [mm]	8	16	26	8	16	26
Maximum relative drift [mm]	8	16	26	8	16	26
Ratio [%]	0.2	0.5	0.8	0.2	0.5	0.8
Differential top drift [mm]	1	1	2	1	1	4
Ratio [%]	0.0	0.0	0.1	0.0	0.0	0.1
Max force roof-roof [kN]	2	5	13	2	5	16
Relative ( )	-	-	-	-	-	-
Max horiz. force roof-beam [kN]	89	112	218	75	84	150
Relative ( )	-	-	-	-	-	-
Max horiz. force beam-column [kN]	101	121	287	79	99	331
Relative ( )	-	-	-	-	-	-
Max force panel-beam [kN]	131	230	312	130	229	310
Relative ( )	-	-	-	-	-	-
Total floor shear [kN]	1875	3153	3402	1860	3132	3372
Relative ( )	-	-	-	-	-	-
Total column shear [kN]	929	1450	1626	923	1436	1614
Relative ( )	0.50	0.46	0.48	0.50	0.46	0.48
Mean column shear [kN]	103	161	181	103	160	179
Relative ( )						
Max column shear [kN]	111	169	188	112	170	189
Relative ( )	1.07	1.05	1.04	1.09	1.07	1.05

Table B4

Response of the 2nd floor - panels with three-connections.

INT3/2nd	y-direction			x-direction		
Quantity	0.18 g	0.36 g	0.60 g	0.18 g	0.36 g	0.60 g
Maximum drift [mm]	19	42	81	20	43	79
Maximum relative drift [mm]	12	26	57	12	27	57
Ratio [%]	0.4	0.8	1.8	0.4	0.8	1.8
Differential top drift [mm]	0	1	3	1	1	3
Ratio [%]	0.0	0.0	0.1	0.0	0.0	0.1
Max force roof-roof [kN]	3	6	13	4	6	14
Relative ( )	-	-	-	-	-	-
Max horiz. force roof-beam [kN]	91	109	157	88	106	141
Relative ( )	-	-	-	-	-	-
Max horiz. force beam-column [kN]	99	132	322	83	129	363
Relative ( )	-	-	-	-	-	-
Max force panel-beam [kN]	165	241	307	166	239	299
Relative ( )	-	-	-	-	-	-
Total floor shear [kN]	1536	2566	2729	1526	2549	2722
Relative ( )	-	-	-	-	-	-
Total column shear [kN]	334	749	1913	324	735	1782
Relative ( )	0.22	0.29	0.70	0.21	0.29	0.65
Mean column shear [kN]	37	83	213	36	82	198
Relative ( )	-	-	-	-	-	-
Max column shear [kN]	52	97	230	51	92	220
Relative ( )	1.40	1.16	1.08	1.41	1.12	1.11

Table B5

Response of the 3rd floor - panels with three-connections.

INT3/3rd	y-direction			x-direction		
Quantity	0.18 g	0.36 g	0.60 g	0.18 g	0.36 g	0.60 g
Maximum drift [mm]	30	65	114	31	66	156
Maximum relative drift [mm]	11	23	36	12	24	84
Ratio [%]	0.4	0.7	1.1	0.4	0.7	2.6
Differential top drift [mm]	1	2	2	2	3	3
Ratio [%]	0.0	0.1	0.1	0.1	0.1	0.1
Max force roof-roof [kN]	9	15	16	10	16	16
Relative ( )	-	-	-	-	-	-
Max horiz. force roof-beam [kN]	90	124	133	110	159	156
Relative ( )	-	-	-	-	-	-
Max horiz. force beam-column [kN]	95	156	194	155	249	255
Relative ( )	-	-	-	-	-	-
Max force panel-beam [kN]	155	259	288	156	261	307
Relative ( )	-	-	-	-	-	-
Total floor shear [kN]	984	1541	1639	976	1538	1652
Relative ( )	-	-	-	-	-	-
Total column shear [kN]	248	479	932	259	506	823
Relative ( )	0.25	0.31	0.57	0.27	0.33	0.50
Mean column shear [kN]	28	53	104	29	56	92
Relative ( )	-	-	-	-	-	-
Max column shear [kN]	34	63	121	37	72	112
Relative ( )	1.23	1.19	1.17	1.29	1.27	1.23

Table B6

Comments

At service limit condition (0.18g), the maximum relative drifts are about 0.2% (that is from 5 to 8 mm) in the 1st storey, and vary from 0.3% to 0.4% (that is from 8 to 12 mm) in the 2nd storey and from 0.3% to 0.4% (that is from 8 to 12 mm) in the 3rd storey. The difference in the maximum relative drifts for panels connected at three and four points is less than 0.1%.

At no-collapse limit condition (0.36g), the maximum relative drifts vary from 0.4% to 0.5% (that is from 13 to 16 mm) in the 1st storey, from 0.6% to 0.8% (that is from 20 to 27 mm) in the 2nd storey and from 0.5% to 0.7% (that is from 15 to 24 mm) in the 3rd storey. The difference in the maximum relative drifts for panels connected at three and four points is less than 0.3%.

At service limit condition (0.18g), the horizontal forces induced to floor-to-floor connections vary from 2 kN to 3 kN in the 1st storey, from 3 kN to 5 kN in the 2nd storey and from 9 kN to 10 kN inr the 3rd storey. The difference in the maximum forces for panels connected at three and four points does not exceed 1 kN.

At no-collapse limit condition (0.36g), the horizontal forces induced to floor-to-floor connections are about 5 kN in the 1st storey and vary from 6 kN to 7 kN in the 2nd storey and from 15 kN to 16 kN in the 3rd storey. The difference in the maximum forces for panels connected at three and four points does not exceed 1 kN.

At service limit condition (0.18g), the horizontal forces induced to floor-to-beam connections vary from 75 kN to 89 kN in the 1st storey, from 88 kN to 95 kN in the 2nd storey and from 90 kN to 113 kN in the 3rd storey. The difference in the maximum forces for panels connected at three and four points does not exceed 8 kN.

At no-collapse limit condition (0.36g), the horizontal forces induced to floor-to-beam connections vary from 84 kN to 112 kN in the 1st storey, from 106 kN to 115 kN in the 2nd storey and from 124 kN to 159 kN in the 3rd storey. The difference in the maximum forces for panels connected at three and four points does not exceed 9 kN.

At service limit condition (0.18g), the horizontal forces induced to beam-to-column connections vary from 79 kN to 103 kN in the 1st storey, from 83 kN to 105 kN in the 2nd storey and from 94 kN to 163 kN in the 3rd storey. The difference in the maximum forces for panels connected at three and four points is up to 22 kN.

At no-collapse limit condition (0.36g), the horizontal forces induced to beam-to-column connections vary from 92 kN to 121 kN in the 1st storey, from 118 kN to 157 kN in the 2nd storey and from 156 kN to 249 kN in the 3rd storey. The difference in the maximum forces for panels connected at three and four points is up to 28 kN.

At service limit condition (0.18g), the horizontal forces induced to panel-to-beam connections vary from 121 kN to 131 kN in the 1st storey, from 142 kN to 166 kN in the 2nd storey and from 128 kN to 156 kN in the 3rd storey. The difference in the maximum forces for panels connected at three and four points is up to 27 kN.

At no-collapse limit condition (0.36g), the horizontal forces induced to panel-to-beam connections vary from 215 kN to 230 kN in the 1st storey, from 237 kN to 241 kN in the 2nd storey and from 194 kN to 261 kN in the 3rd storey. The difference in the maximum forces for panels connected at three and four points is up to 66 kN.

The results show that the practically symmetrical (except of the floor element arrangement) load bearing structural system leads to similar response of the building in the two directions. All quantities tabulated above, such as the maximum top displacement and the base shear, have negligible differences for loading along the X- and Y-direction. Additionally, the small in-floor differential displacements imply that the selected floor configuration (TT elements connected with each other at three points) acts as a rigid diaphragm.

In general, it can be said that the response of the structure with panels connected at four points shows similar characteristics with the one with panels connected at three points. As expected, the building with four-point connected panels, being stiffer than the building with three-point panels, shows larger values of the base shear force and lower values of the top displacement.

ANNEX C - ANALYSES OF MULTI-STOREY BUILDINGS WITH ISOSTATIC CONNECTIONS

The plan, the section and the side view of the analysed multi-storey building are presented in Figures C1, C2 and C3 respectively. The dimensions of the floor plan measured between the centres of the corner columns are 14 x 14 m. The structure is symmetrical – there are two 7m spans in both orthogonal directions. The height of the first storey is 3,3 m, the height of the second and third storey is 3,2 m. The total height of the structure measured from the bottom to the axis of the top beam is therefore 9.7m.

Structures are supported by 9 square reinforced concrete columns. The cross-section of columns is presented in Figure C4. The floors and the roof consist of precast pre-stressed concrete slabs. It is assumed that these elements provide a completely rigid diaphragm.

3D numerical models for two different types of the described multi-storey building are elaborated. The first type is characterized by the hinged beam-column connections and the second type by the semi-rigid beam-column connections. Modelling of these connections is described in the following section.

In the same structure, horizontal and vertical panels were used, as presented in Figure C3. At first, continuous vertical panels from the bottom to the top of the building were planned. Then these panels have been divided into three parts to prevent impact between the horizontal and vertical panels). In this way, also large forces in the panel-to-structure connections due to higher modes were avoided.

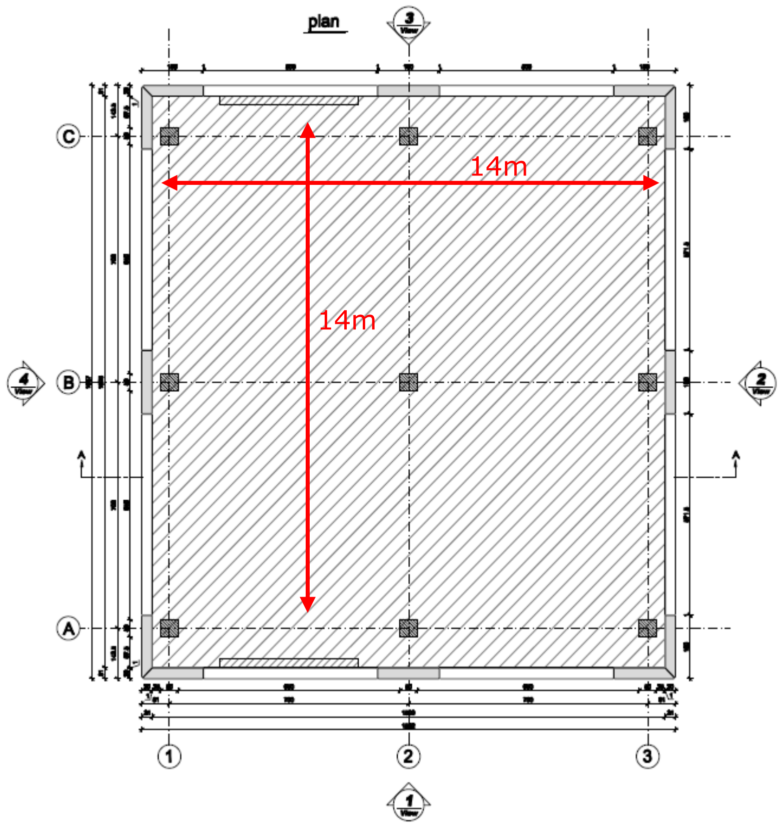


Figure C1: Plan of the building

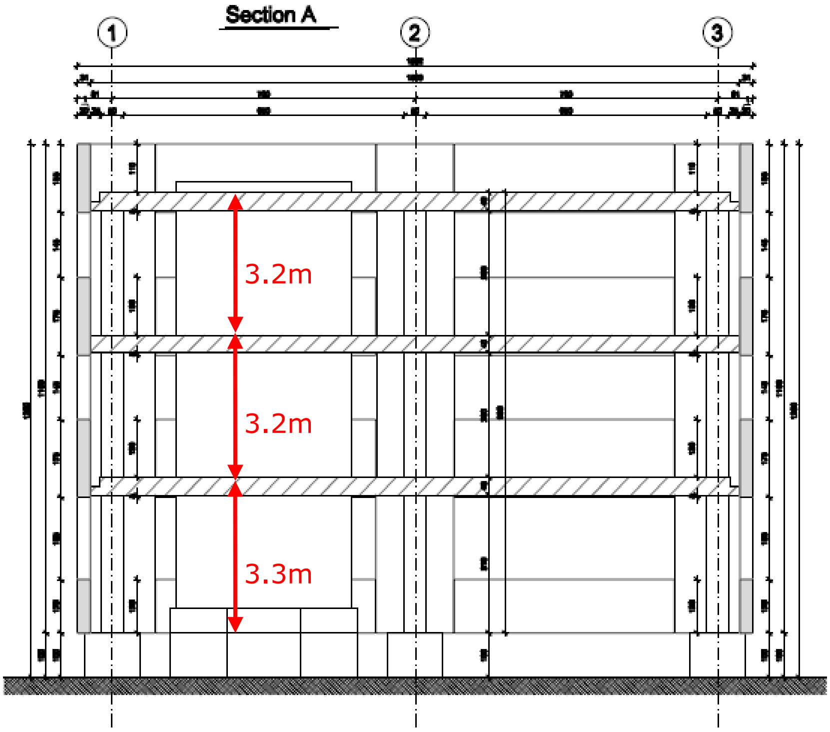


Figure C2: Section of the building

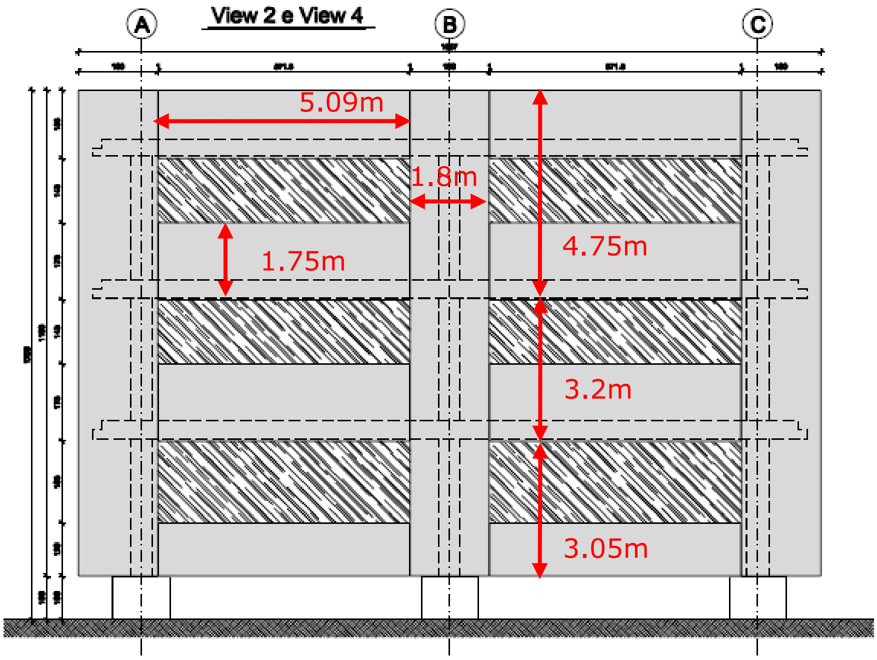
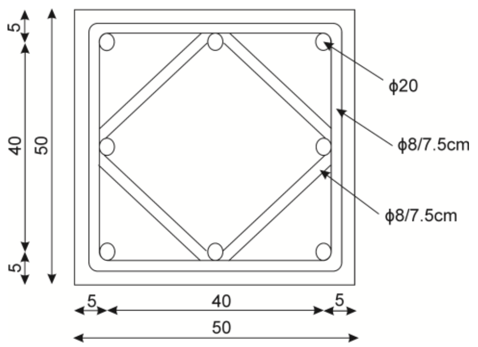


Figure C3: Side view of the building



STEEL CLASS B450C

CONCRETE CLASS C45/55

LONGITUDINAL BARS Φ 20

STIRRUPS Φ8/7.5 cm

DISTANCE OF THE LONG. BARS FROM THE EDGE 5cm

Figure C4: Cross-section of the column

The dimensions of the vertical and horizontal façade panels are presented in Figure C3. The thickness of all panels is 30 cm. As in the case of one-storey buildings, they have been modeled using standard elastic beam-column elements (see Figure C5). The flexural stiffness corresponding to the gross cross-section has been reduced by 50% to take into account panel cracking. Modulus of elasticity has been defined based on the class of concrete C45/55. The strength of the panels is not limited. No interaction between individual the panels is taken into account.

At their lower and upper ends, vertical panels are attached to the structure with pinned connections. In this way, the panels can rotate without any restrain when horizontal loading is applied to the building. The forces in the panel-to-structure connections are therefore equal to the floor acceleration at the level of the connection multiplied by the mass of the panel.

For the horizontal panels it is assumed that they are completely fixed to the floors and therefore they follow the structure as a rigid body. This assumption allowed modelling of the horizontal panels simply as additional masses at inter-storey levels.

As already mentioned, it is assumed that the floor elements together with the connections between the floor elements and main beams were stiff enough to provide rigid diaphragm. More attention is paid to the connections between beams and columns. Structures with two different types of beam-column connections are analysed: a structure with hinged connections (Figure C6a) and a structure with semi-rigid connections (Figure C6b). Nonlinear response of semi-rigid connections is presented in FigureC6b. At first, the moment in the connection is low. After the gap closes the rotational stiffness increases until yielding of the longitudinal reinforcement in the beams. Such behaviour of the semi-rigid connections is calibrated using the results pseudo-dynamic tests performed in the frame of SAFECAST project (see deliverable SAFECAST WP5.2 - Calibration of the connections for more details).

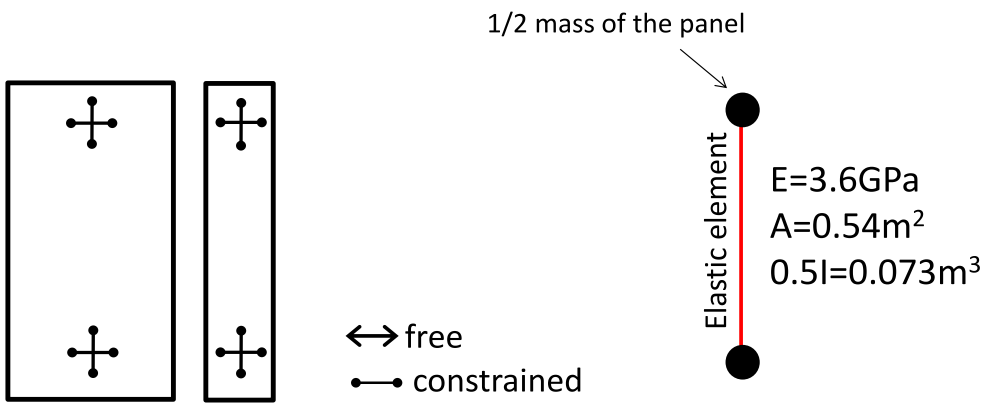
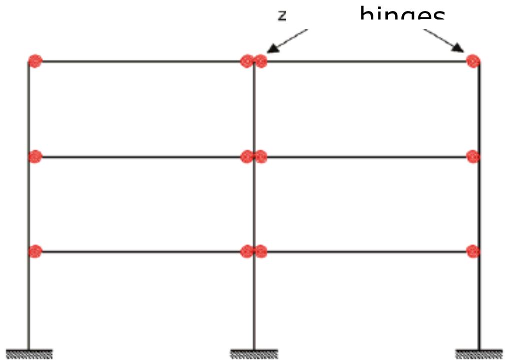
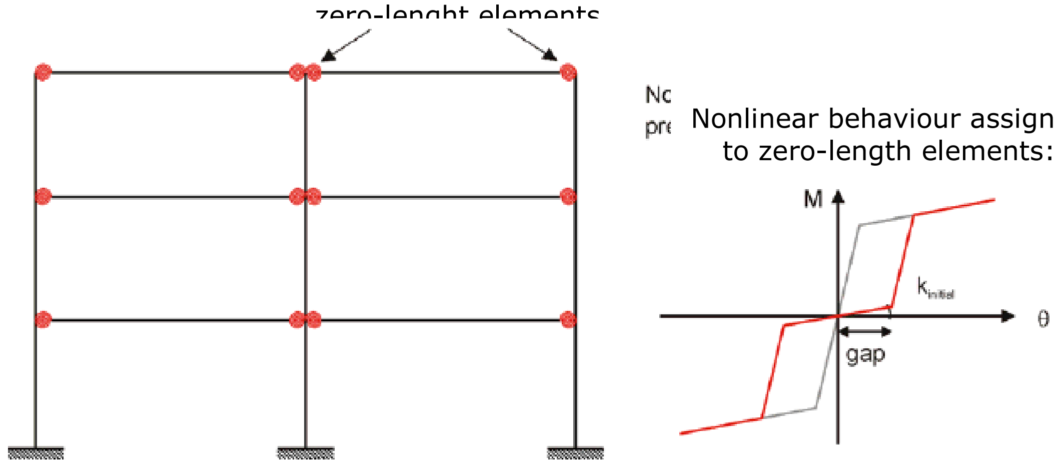


Figure C5. Geometry and numerical model of vertical panels



(a) Structure with hinged beam-column connections



(b) Structure with semi-rigid beam-column connections

Figure C6. Modelling of the (a) structure with hinged and (b) semi-rigid beam-column connections

Columns, with the cross-section presented in Figure C4, are modelled using distributed plasticity elements as in the case of one-storey buildings. Since the floor and roof elements are rigid enough to provide a rigid diaphragm, the floor mass is not distributed along the elements but it is concentrated in single mass points at each storey which are located in the centre of the plan. The mass of the first, second and third floor is 1677 t, 1548 t and 1246 t respectively.

Additional mass points deriving from the horizontal panels are added at each floor. There are eight horizontal panels per floor, each weighing 6.8t. The mass of the vertical panels is modelled as illustrated in Figure C6. The mass is concentrated at the bottom and top end of the panel, half on one and half on the other side.

For the nonlinear history analysis, modified Tolmezzo accelerogram (Figure C7a) was used. It was modified to fit the Eurocode 8 spectrum (Figure C7b) for soil type B. Three PGA (peak ground acceleration) intensities were taken into account: 0.18g (corresponding to 0.15g for soil type A), 0.36g (corresponding to 0.3g for soil type A) and 0.6g (corresponding to 0.5g for soil type A).

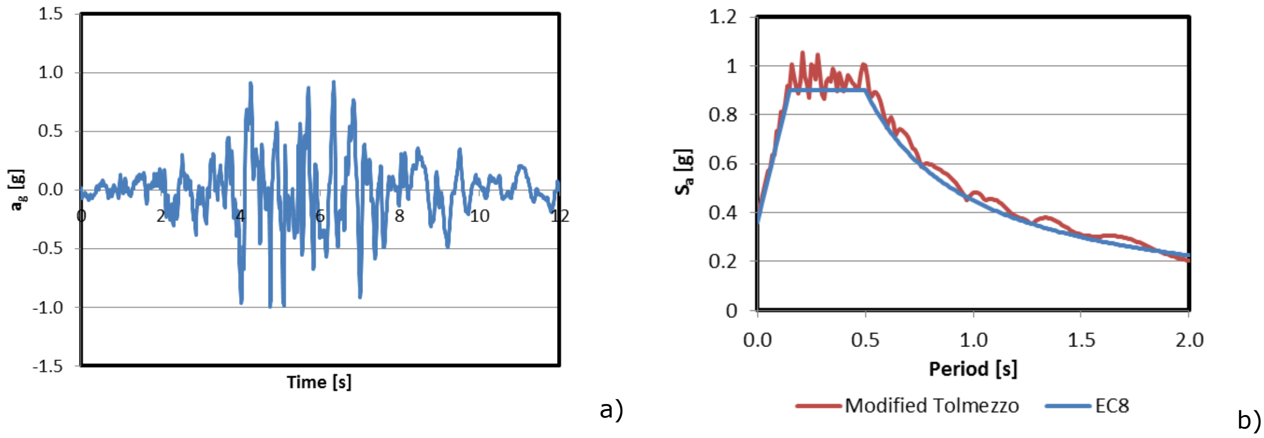
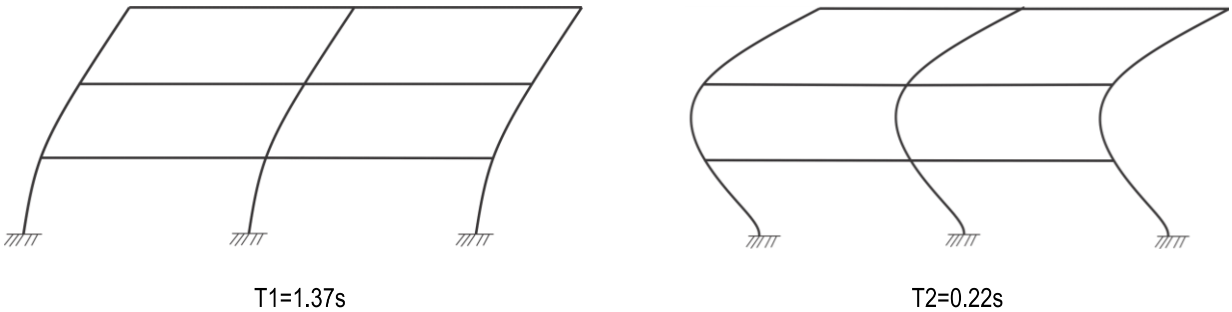
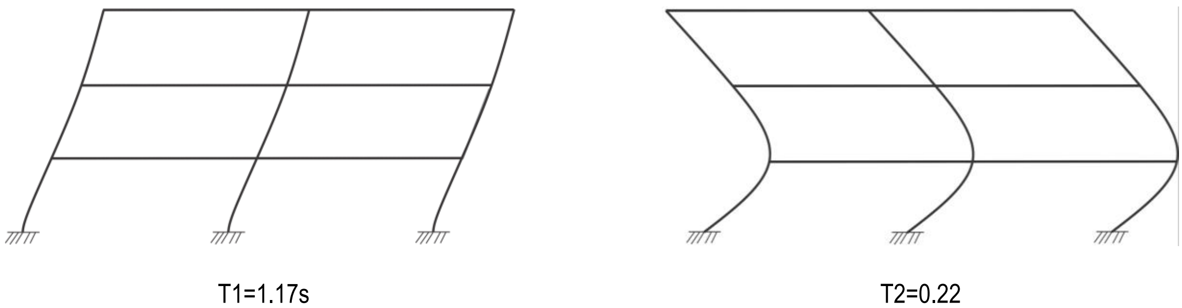


Figure C7: Modified Tolmezzo accelegram (a) and corresponding spectrum (b)



(a) Structure with hinged connections



(b) Structure with semi-rigid connections

Figure C8: Modes and periods of vibration for multi-storey structures

The side view of the first two modes of vibration of the two analyzed multi-storey buildings is presented in Figure C8. In the case of hinged beam-column connections (Figure C8a), the first mode of vibration reflects typical cantilever behavior, while in the case of semi-rigid beam-column connections (Figure C8b), the response is more similar to a frame system. First period of the structure with semi-rigid connection is not much higher than the period of the structure with hinged connection. This is due to the initial gap in the semi-rigid connections.

The results for buildings with isostatic arrangement of facade panels and hinged or semi-rigid beam-column connections are given in Tables C1 and C2.

Response quantities for the multi-storey building with hinged beam-column connections

	PGA = 0.18 g	PGA = 0.36 g	PGA = 0.60 g
Maximum top drift (mm)	97	165	294
Ratio (%)	1.0	1.7	3.0
Maximum storey drift (mm)	42	67	113
Ratio (%)	1.3	2.0	3.4
Max force beam-column (kN)	48	50	71
Total column shear (kN)	3459	4292	4487
Mean column shear (kN)	384	477	499
Max column shear (kN)	580	664	695
Ratio (%)	120.8	119.9	134.3
Max force panel-structure (kN)	14	14	21

Table C1

Response quantities for the multi-storey building with semi-rigid beam column connections

	PGA = 0.18 g	PGA = 0.36 g	PGA = 0.60 g
Maximum top drift (mm)	169	216	378
Ratio (%)	1.7	2.2	3.9
Maximum storey drift (mm)	82	103	162
Ratio (%)	2.5	3.1	4.9
Max force beam-column (kN)	32	67	71
Total column shear (kN)	3331	3952	3994
Mean column shear (kN)	370	439	444
Max column shear (kN)	493	667	670
Ratio (%)	133.2	151.8	150.9
Max force panel-structure (kN)	9	18	21

Table C2

ANNEX D - ANALYSES OF MULTI-STOREY BUILDING WITH DISSIPATIVE CONNECTIONS

The plan and the section of the analysed multi-storey building have already been given in Appendices B and C, and will not be repeated here. The elevation where the panels can be seen is given in Figure D1. The dissipative connections presented here can work under high level of shear deformations (i.e. one side of the cushion moving in the opposite direction of the other side), thus the vertical panels are used in energy dissipation only (see Figure D2). The horizontal panels are implemented into the model as mass for the sake of consistency with the other analyses by isostatic and integrated panel working groups.

The dimensions of the floor plan measured between the centres of the corner columns are 14 x 14 m. The structure is symmetrical – there are two 7m spans in both orthogonal directions. The height of the first storey is 3,3 m, the height of the second and third storey is 3,2 m. The total height of the structure measured from the bottom to the axis of the top beam is therefore 9.7m.

Structures are supported by 9 square reinforced concrete columns. The cross-section of columns is presented in Figure D3. The floors and the roof consist of precast pre-stressed concrete slabs. It is assumed that these elements provide a completely rigid diaphragm.

The structure is analysed by using hinged beam-to-column connections. The columns are modelled as nonlinear elements while the beams and panels are elastic in the numerical model.

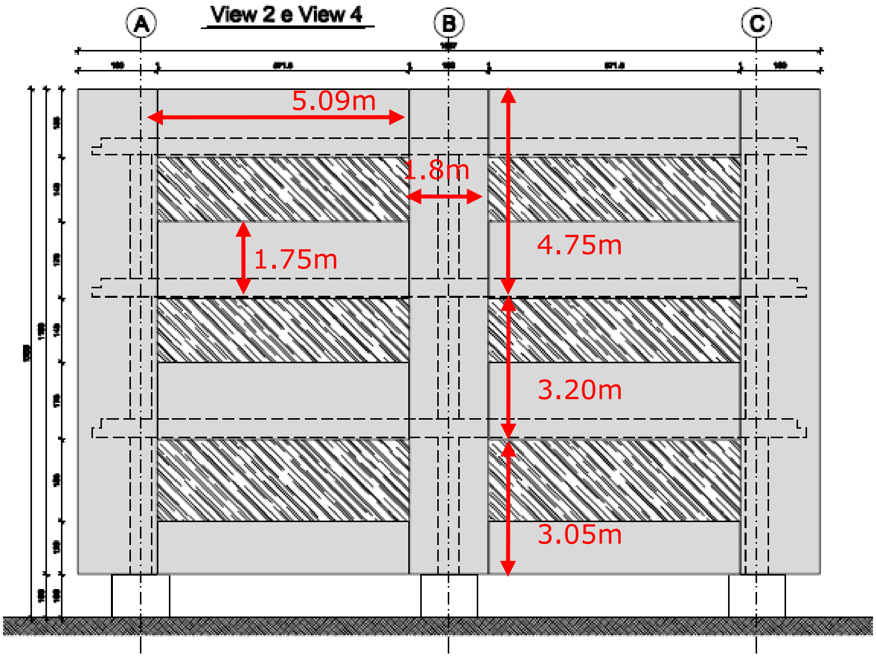


Figure D1: Side view of the building

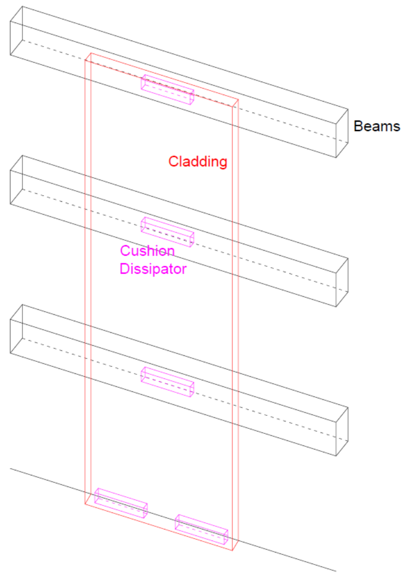
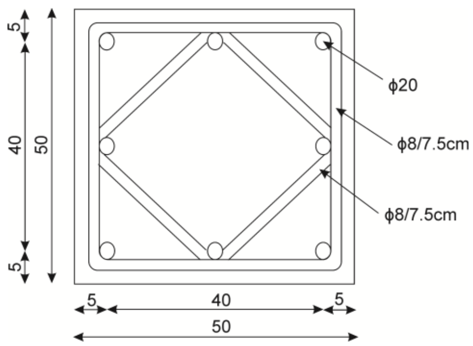


Figure D2: Cross-section of the column



STEEL CLASS B450C

CONCRETE CLASS C45/55

LONGITUDINAL BARS Φ 20

STIRRUPS Φ8/7.5 cm

DISTANCE OF THE LONG. BARS FROM THE EDGE 5cm

Figure D3: Cross-section of the column

The thickness of all panels is 30 cm. The flexural stiffness corresponding to the gross cross-section has been reduced by 50% to take into account panel cracking. Modulus of elasticity has been defined based on the class of concrete C45/55.

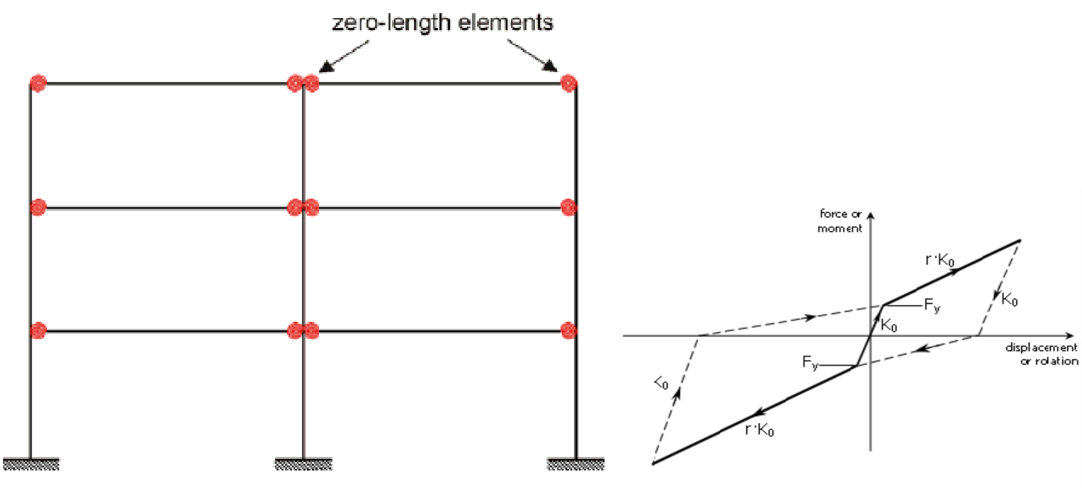


Figure D4. On the left is the frame model of the structure with hinged beam-column joints and on the right is the force-deformation loop used for the plastic dissipators

Columns are modelled using distributed plasticity elements with force-based formulation. Five integration points are used per column member. The floor masses are distributed over the beams. Panel masses are defined on the panel elements as distributed mass.

For the nonlinear history analysis, modified Tolmezzo accelerogram was used as shown in previous appendices. The PGA (peak ground acceleration) used in the analyses is 0.18g, 0.36g (corresponding to 0.3g for soil type A) and 0.60g.

The response quantities of the structure are given in Table D1. As compared to the isostatic structure, given in Appendix C,

- the top displacement decreased to around 100%
- the maximum storey drifts decreased almost 100%
- the total column base shear decreased 25%
- higher decrease in displacements may be attributed to the increase of overall equivalent damping

Response quantities for the multi-storey building with hinged beam-column connections

	PGA = 0.18 g	PGA = 0.36 g	PGA = 0.60 g
Maximum top drift (mm) Ratio (%)	34 0.35	72 0.74	169 1.74
Maximum storey drift (mm) Ratio (%)	14 0.43	31 0.9	62 1.9
Max force beam-column (kN)	10	22	51
Total column shear (kN)	1928	3255	4108
Mean column shear (kN)	214	362	456
Max column shear (kN) Ratio (%)	298 139	488 135	562 123
Max force panel-structure (kN)	42	60	60

Table D1

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