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Structural assessment of modular precast 3D cell mid- to high-rise buildings with different connections

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Abstract

Precast construction employing modular 3D cells for housing was developed alongside frame and panel buildings since the end of WWII, mainly in Europe. This technology combined with in-situ concreting of wet joints was employed with a certain success throughout Europe up to the '80s, after which it became progressively less popular due to the difficulties in handling transportation (both lifting and shipping due to the large cell dimensions) and limited benefits in construction due to the partial prefabrication, framing its modern application in many countries to relatively small-size building components, such as kitchen/bathroom or service blocks. Thanks to the recent innovations of the precast concrete technology (both in production and structural connections), combined with the market evolution, this technology is nowadays experiencing a renovated interest for mid- and high-rise buildings, especially in Asia, where rapid dry or semi-dry assemblage of the cells ensures the full finishing of the units in factory, and the full exploitation of the benefits induced by the prefabrication process. As a matter of fact, the current literature regarding the structural behaviour of buildings employing this technology is lacking from a robust assessment, especially concerning their seismic performance. As a preliminary attempt to fill this gap, this paper presents the results of traditional seismic analysis with response spectrum carried out on a representative large residential building designed having 6, 12, 18 and 24 storeys modelled with shell elements and spring connections, analysing the limit PGAs associated to each typology and commenting the role of different connection devices and the possible design implications.

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Keywords: 3D cell; modular construction; precast; seismic assessment; joint; connections.

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1. Introduction

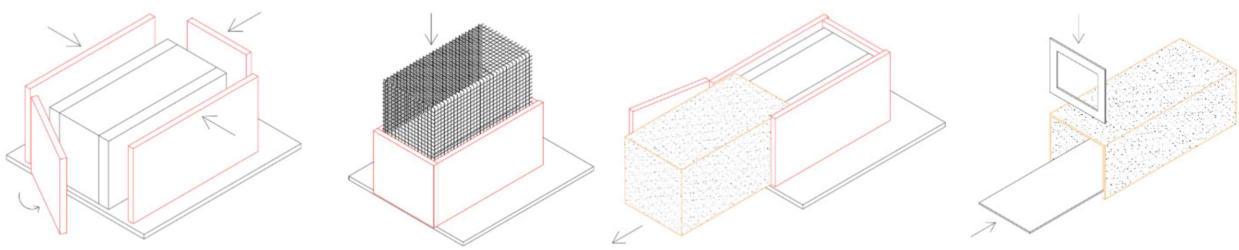
The precast concrete construction industry sector devoted to building structures delivers manufacts which may be classified according to the dimensions of the elements as: (I) frame elements - monodirectional beams; (II) wall elements - bidirectional panels; (III) cell elements - 3D units (EN 1998-1:2004). Cell structural elements, far less popular than frame or wall ones, were employed since the 1950's in Europe for the construction of modular buildings devoted to residential housing (Koncz et al. 1979, IASM 1986). Their diffusion in Europe encountered a setback around the end of the 1980's, mainly due to the combination of the difficulties in handling transportation (both lifting and shipping) of the heavy and bulk precast cells and the only limited benefits provided by their partial prefabrication. Indeed, in this period the mutual connections of the cells were made by wet joints with protruding rebars encased into cast-in-situ concrete. So far, 3D cell elements are currently mainly produced regarding non-structural elements, such as bath/kitchen technological blocks, or service blocks for the distribution of MEP systems.

Thanks to the recent developments in the field of precast concrete construction in terms of both connection mould technology, allowing for dry-assembled or semi-dry-assembled (with small volumes of mortar replacing large volumes of concrete) structural bodies with mechanical high-precision connections, the precast modular construction employing 3D cell units is currently experiencing a renovated boost (Lawson & Richards 2010, Knaacl et al. 2012, Yang 2021), especially in the Asian markets, where several examples or large residential blocks as tall as almost 30 storeys have recently been built or are currently under construction employing this technology.

Nevertheless, the current literature regarding the structural behaviour of buildings employing this technology is lacking from a robust assessment, especially concerning their seismic performance. With the aim to preliminarily fill this gap, this paper presents a numerical study carried out with reference to 4 representative large residential buildings having 6 – 12 – 18 – 24 storeys employing a specific 3D cell technology. After preliminary architectural and distributive design, the building structures were numerically modelled and designed according to typical static load combinations, and later checked against seismic load with modal and response spectrum analysis, deriving the limit PGAs associated to each typology. The analysis considered the role and the possible design implications of different dry- or semi-dry mutual connection devices between the 3D cell units.

2. 3D cell precast system at study

There are several different techniques to produce 3D cell elements, which are strictly related to the mould technology and have large impact over the architectural composition and final aspect. Typically, the moulds allow to produce monolithic elements with 4 walls out of the total 6, although more complex moulds allow for the production of up to 5 walls at once. The technology considered in the present study is based upon a 3D mould with collapsible core allowing for the production of single monolithic cells with one short side, both long sides, and the soffit. Each modulus is completed in the production factory with the installation of a bottom ribbed concrete plate and the remaining short side façade wall, both separately precast. The production steps are sketched in Fig. 1. To be noted that this technology ensures the full finishing of the cells within the production factory, and a very fast assemblage on the construction site, with the specific MEP systems already installed during casting of the unit and both vertical and horizontal MEP distribution occurring in designated service volumes.



(a) (b) (c) (d)

Fig. 1. Production steps of the 3D cell precast element: (a) preparation and closing of the mould; (b) insertion of reinforcing cage with inserts; (c) concrete casting and extraction of the element after hardening; (d) installation of separately precast bottom slab plate and façade elements.

3. Connection devices

Modern joint detailing, deriving from the experience of dry- or semi-dry-assembled frame or wall precast systems, is considered as an alternative to traditional wet joints with protruding rebars conglobated into concrete casting. The vertical connection of the 3D cell units, as shown in Fig. 2, is made with over-designed bolted mechanical couplers which, alternatively to typical wall shoes, allow for the full development of plastic elongation of the rebars installed in both upper and lower units, thus maintaining the typical ductility and energy dissipation capacity of emulative joints (Dal Lago et al. 2016, 2021b). The coupler plate, installed in the upper element, can be placed in many required positions along the perimeter of the unit, as well as the protruding threaded bars inserted in the lower element. The joint requires a small amount of mortar to be poured for completion, after threaded bars are bolted.

Different mechanical devices were considered for the horizontal connection between units, as shown in Fig. 3: (a) dowelled plate, where the vertical threaded bars protruding from the bottom unit for the vertical connection are inserted into a thick steel plate with oversized round holes to allow for tolerances, which are then compensated with the use of slotted washers, filled with mortar when the vertical joint is completed; (b) diamond-loop mortar joints (Dal Lago et al. 2021a), where diamond-shaped recesses left in the adjacent vertical walls of the units are reinforced with flexible high-strength steel loops, which confine a centrally inserted rebar after mortar is poured to complete the joint; (c) welded plate (Dal Lago et al. 2022), where two anchored angles are cast in the adjacent vertical walls of the units, and a flat steel plate is placed and welded peripherally on site.

The horizontal connections are also fundamental to provide strength and stiffness to the diaphragmatic action (Dal Lago & Ferrara 2018, Dal Lago et al. 2019), since a cast-in-situ concrete topping is not employed.

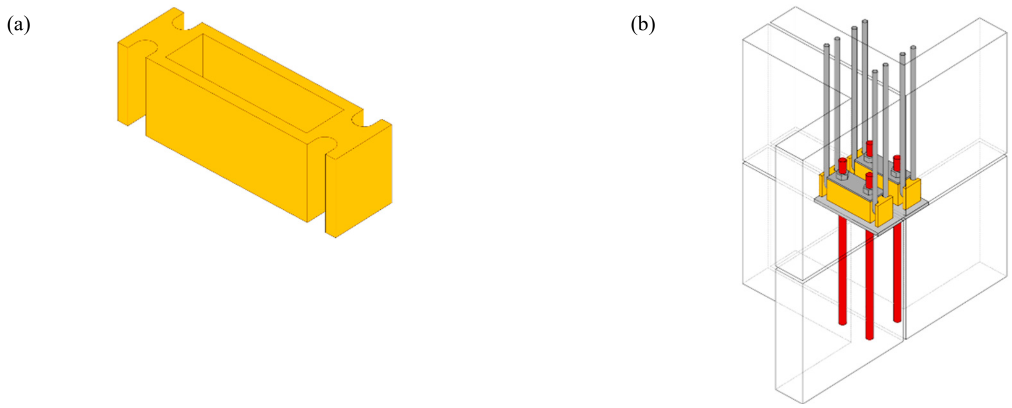


Fig. 2. Conceptual sketch of the vertical connection system: (a) coupling plate cast in the upper element; (b) assembled joint.

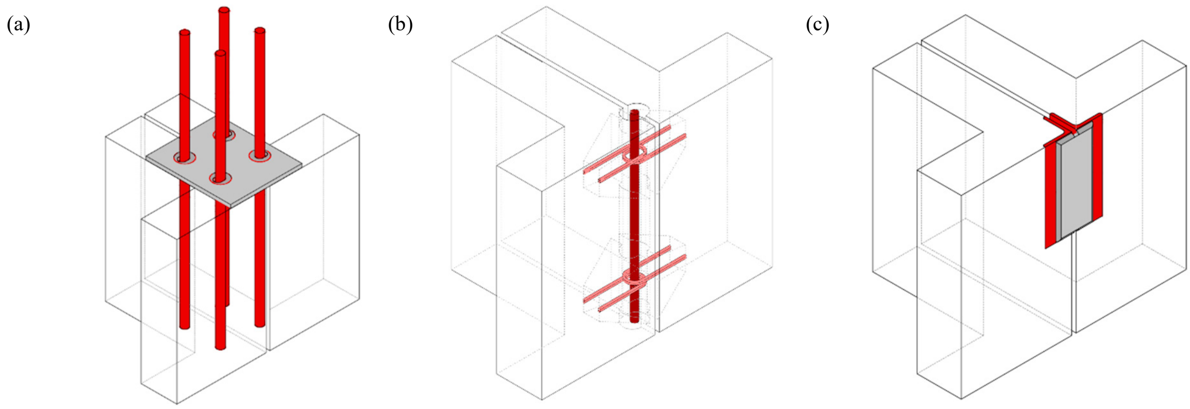


Fig. 3. Conceptual sketch of the horizontal connection system: (a) dowelled plate; (b) diamond-loop mortar joint; (c) welded plate.

4. Case study buildings

The modular system at study produces elongated 3D cells, as typical for this technology due to constraints in transportation width, with possibility of large openings and balcony cantilevers in the façade. A typical layout is proposed in Fig. 4, with reference to a building having plan dimensions of 42.9m x 20m. Mirrored stair/elevator blocks are located at the external of the short side of the building. The units have fixed width of 3.2m, fixed height of 3.2m, and variable length, the maximum being 9.0m (Fig. 5). Four different buildings are considered in the analysis, being an aesthetically-appealing combination of duplex apartments for 4 persons (4P) and standard flats for 4, 2, and 1 persons (4P, 2P, 1P, respectively). The buildings have different number of storeys of 6 (height 19.2m), 12 (height 38.4m), 18 (height 57.6m), and 24 (height 76.8m). To be noted that the considered buildings are addictive, meaning that the top 6 storeys of all buildings are all equal to the 6-storey buildings, and so on. Pre-proportioning of the units under static loading defined the thickness of the vertical panels of the standard residential units equal to 10cm, with the exception of the base 6 storeys of both the 18- and the 24-storey buildings, enlarged to 15cm. The units for the vertical connectivity (external cores) have all thickness equal to 15cm. The weight of every single assembled unit was kept below the limit of 20t. Concrete class C40/50 and steel grade B450A/C for ordinary reinforcement were considered.

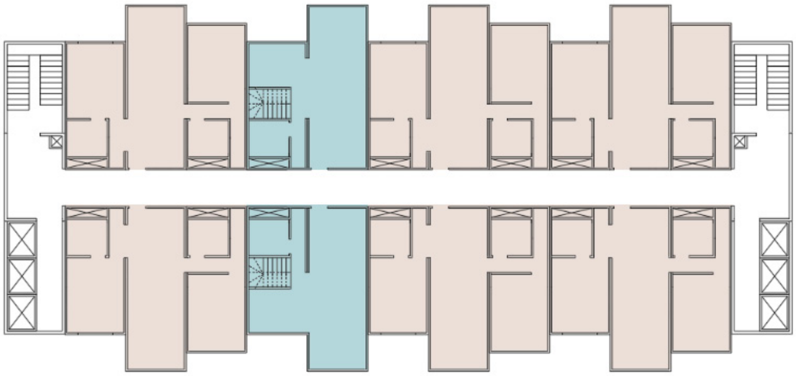


Fig. 4. Typical floor of the considered buildings.

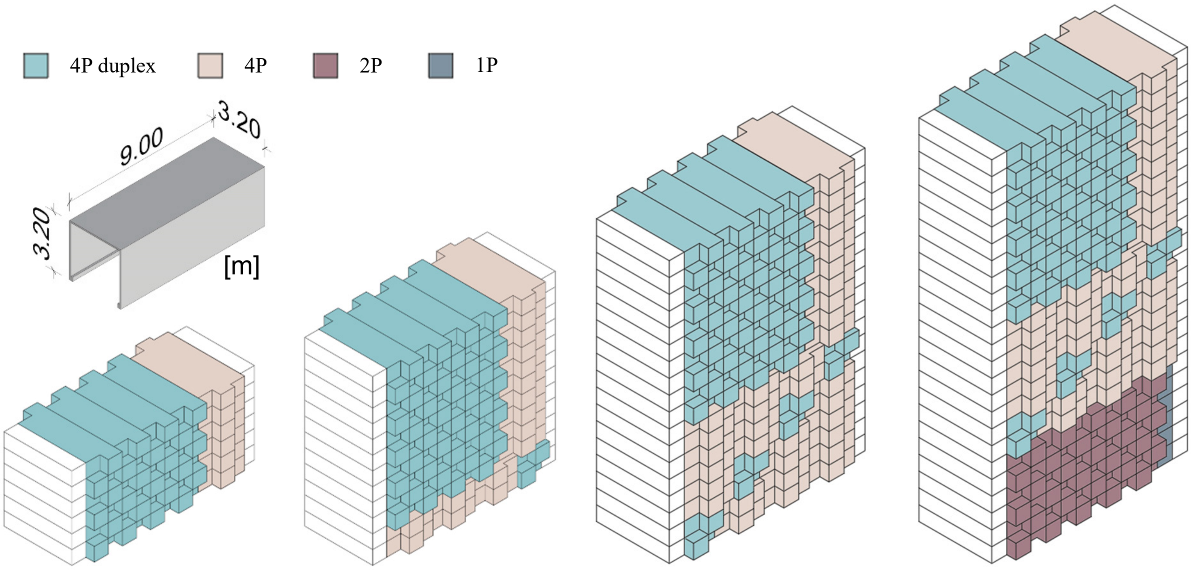


Fig. 5. Case study buildings with 6 – 12 – 18 – 24 storeys with distribution of apartments and maximum dimensions of the 3D cell unit.

5. Numerical models

Numerical models were built with the finite element software Straus7. The structural walls of each 3D cell unit were modelled with shell elements with linear shape functions, as shown in Fig. 6 with reference to a typical floor. The bottom nodes were fully fixed to the ground, simulating a perfect foundation system. The vertical connections were assumed to be emulative, following what previously described, and thus were not explicitly modelled. The lateral mutual connections between the units were introduced with horizontal spring connection elements located at the top of the simulated moduli in the designed positions. Their elastic stiffnesses, reported in Table 1, were attributed based on the experimental evidence collected in the literature, including Dal Lago et al. (2019, 2021a, 2022).

The walls which are not explicitly part of the lateral load resisting system, e.g. façade panels and bottom slabs with cantilevering balconies, isostatically connected to the units, as well as the landings and corridor plates, were introduced as load patches in terms of masses without stiffness. Both gravity and lateral wind loads were also introduced based on the location of Milan, Italy, through load patches, including the dynamic factors to convert them in seismic masses. The whole design process, including definition of loads and combinations, was based on the current Italian standards.

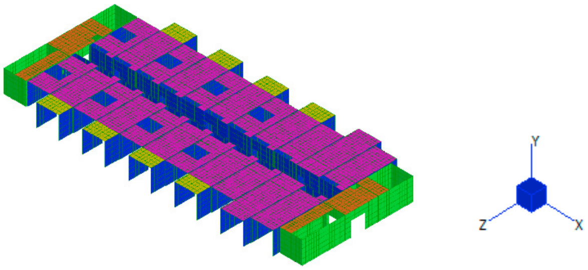


Fig. 6. 3D FEM model of a typical floor composed by shell elements.

Joint	Axial Horizontal [KN/m]	Shear Horizontal [KN/m]	Shear Vertical [KN/m]
dowelled plate	1×10^4	1×10^4	1×10^3
diamond-loop mortar joint	1×10^6	1×10^6	1×10^6
welded plate	2×10^5	2×10^5	2×10^5

Table 1. Stiffness values attributed to the connection elements.

6. Dynamic properties

The main vibrational modes extracted from the models of the 24- and 6-storey buildings are shown in Fig. 7 and Fig. 8, respectively, for the three proposed connection solutions.

(a) (b) (c)

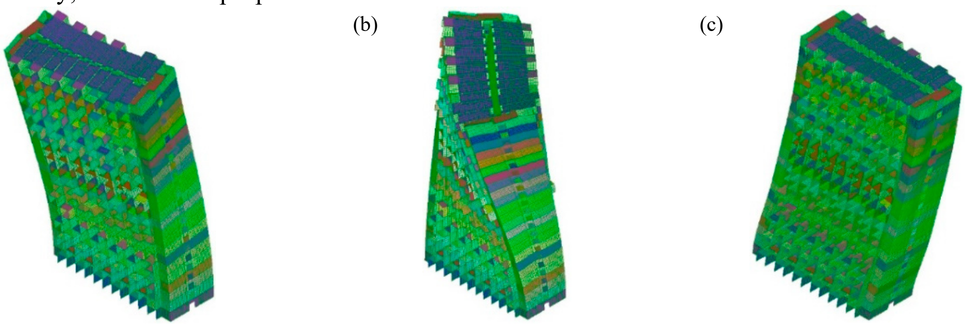


Fig. 7. Main modal shapes for the 24-storey building with diamond-loop mortar joints: (a) 1st mode – translational in Z-dir; (b) 2nd mode – torsional around Y-dir; 3rd mode – translational in X-dir.

(a) (b) (c)

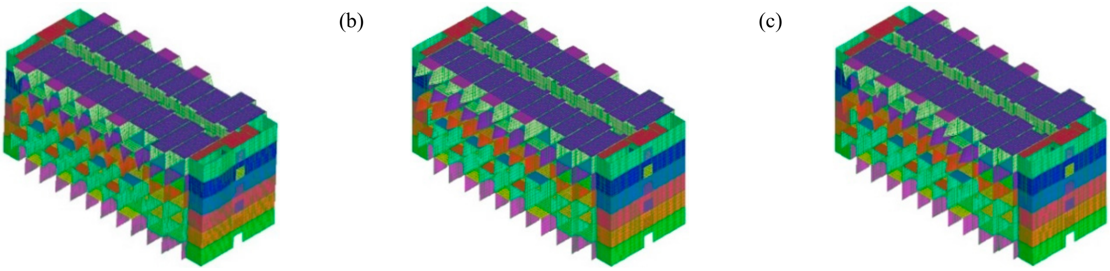


Fig. 8. Main modal shapes for the 6-storey building with diamond-loop mortar joints: (a) 1st mode; (b) 2nd mode; 3rd mode. All modal shapes are characterised by local deformation of the cell units.

Whilst the deformed shapes of the main modes of the 24-storey building clearly suggest a global deformation trend, those of the 6-storey building are associated with local out-of-plane deformations of vertical unit walls. This is reflected in the dynamic properties of the buildings, resumed in Fig. 9, where it can be observed that there are several modes detected by the analysis having similar periods, which are associated to a very low increase of participating mass. This makes the attainment of a large cumulated mass relatively difficult, especially for squatter buildings, unless even hundreds of modes are considered in the analysis.

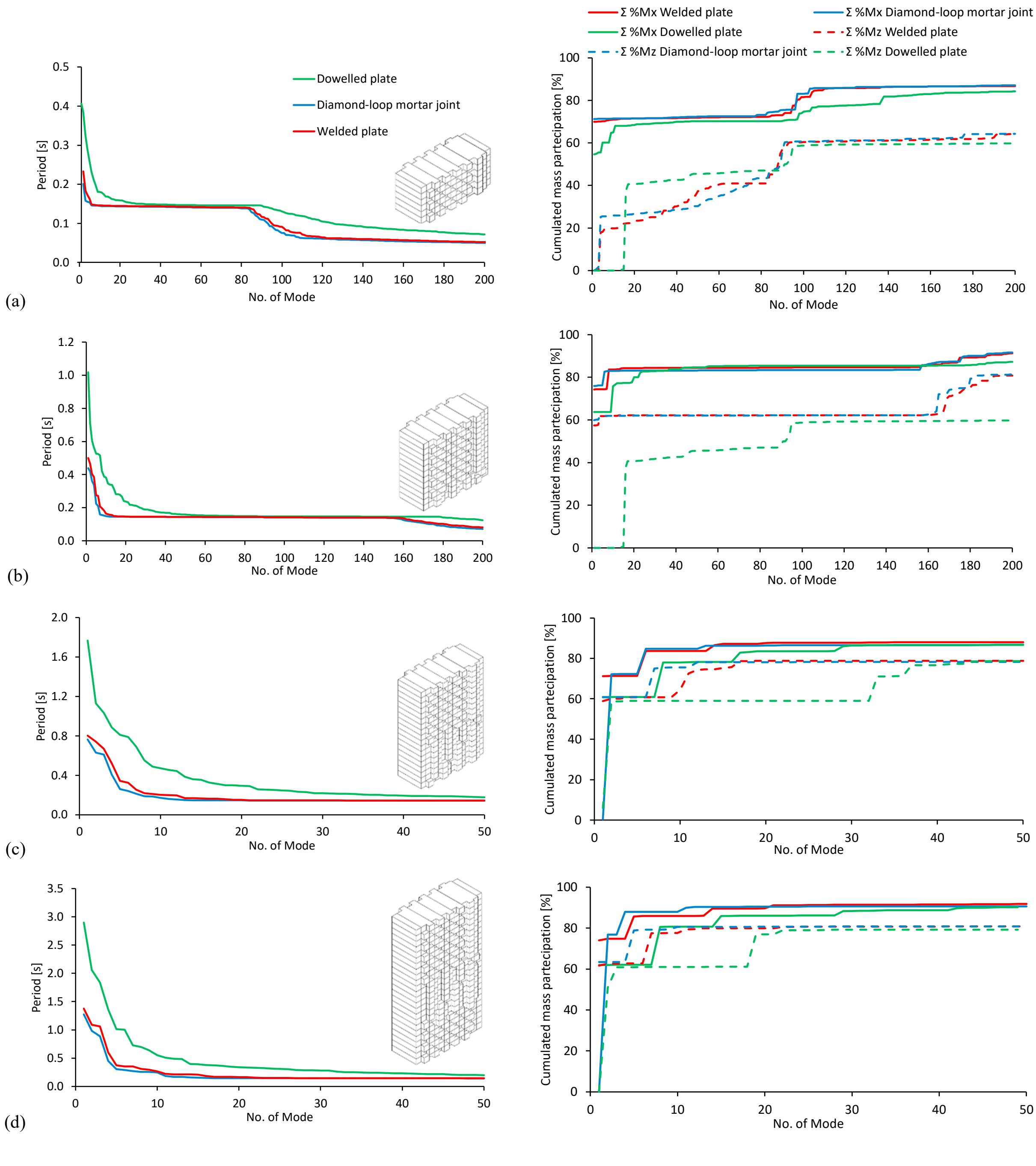


Fig. 9. Dynamic properties in terms of period and cumulated participating mass for the case study structures having: (a) 6 storeys; (b) 12 storeys; (c) 18 storeys; (d) 24 storeys.

7. Seismic analysis with response spectrum and simplified check criteria

The seismic check of the buildings under consideration was carried out by means of modal analysis with response spectrum, given the dynamics of the system is far from being predominantly associated to a single mode, thus excluding both linear and non-linear static analysis (Dal Lago & Molina 2018). Combined horizontal elastic spectra as per Eurocode 8 for subsoil type B were applied, leaving specific considerations about the adoption of anelastic spectra to the post-processing analysis. Examples of principal stress distributions are shown in Fig. 10 and Fig. 11 for the 24- and 6-storey buildings, respectively, with reference to one connection type and load combination.

Actions on connections were also checked and were found not to be critical.

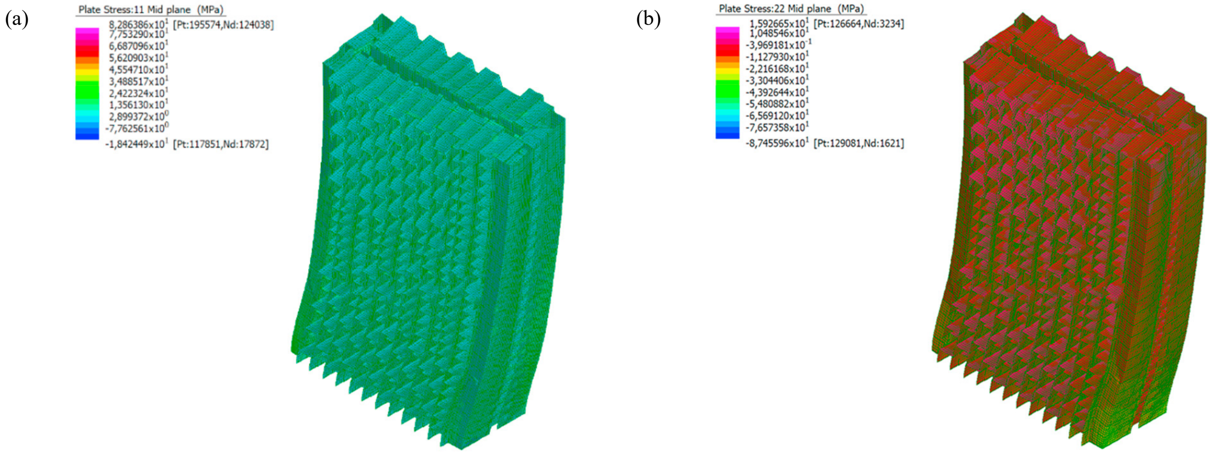


Fig. 10. Deformed shape and principal stresses for the 24-storey building with diamond-loop mortar joints under seismic combination with PGA=0.2g in X-dir (elastic spectrum): (a) tension; (b) compression.

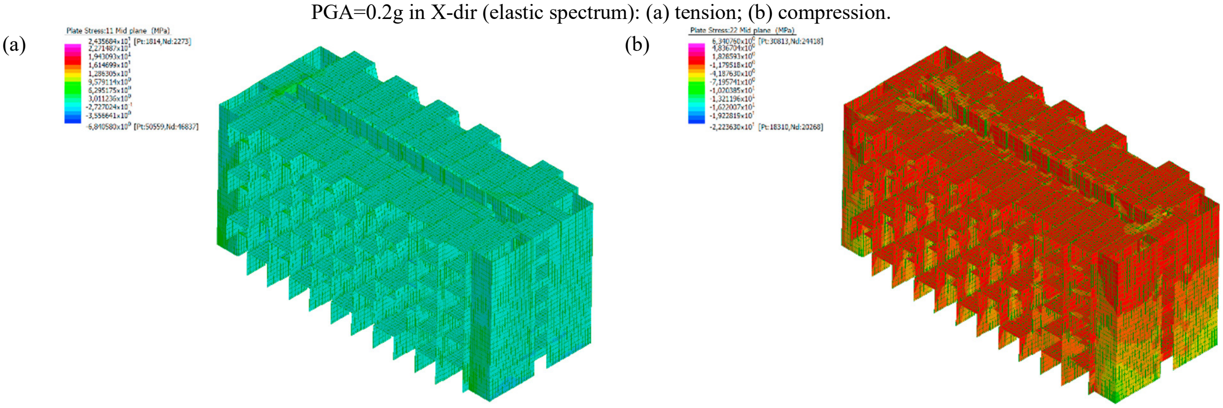


Fig. 11. Deformed shape and principal stresses for the 6-storey building with diamond-loop mortar joints under seismic combination with PGA=0.2g in X-dir (elastic spectrum): (a) tension; (b) compression.

A simplified resistance criterion of the concrete cell walls was adopted, concerning the check of principal stresses, contemporarily setting: (a) the maximum compression stress to the design concrete strength, leading to 24 MPa, associated to either unreinforced concrete C40/50 or concrete C35/45 with 1% B450C reinforcement and (b) the maximum tension stress to 16 MPa, assuming the maximum geometric reinforcement ratio, equal to 4%. Response spectrum analyses were carried out considering elastic spectra having PGAs of 0.1g, and 0.2g, later deriving a critical stress domain by linearly connecting the stress values associated to the seismic analyses with that in static load condition (Fig. 12a). Charts summarising the results of all analyses, encompassing different building heights, connections, and behaviour factor, are reported in Fig. 12b and Fig. 12c considering the maximum stress of all shell elements (σ_{max}), and to the 100th more stressed shell element (σ_{100}), respectively, acknowledging the capacity of increasing locally the reinforced concrete resistance by applying specific measures in a limited number of locations.

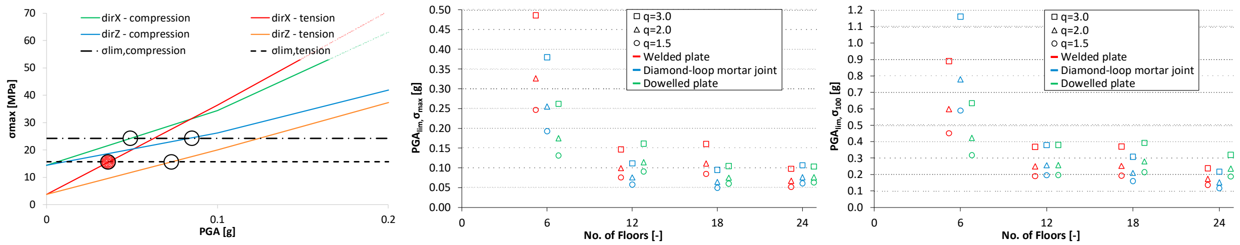


Fig. 12. Limit PGAs: (a) identification procedure of a single case with elastic spectral analysis; (b) chart referred to σ_{max}; (c) chart referred to σ_{100}.

8. Conclusions

The frequency analysis of 3D cell modular buildings requires a large number of modes to be considered, even hundreds, due to the influence of the out-of-plane vibration of the vertical unit walls. The period of the main modes strongly depends upon the typology of connections, with those associated to dowelled plates, the most deformable, being about the double. The limit PGA found according to simplified maximum stress criteria after response spectrum analysis is decreasing relevantly when passing from the 6- to the 12-storey building, and small variations are observed for taller buildings, with a slightly decreasing trend with the height, associated to a balance between larger flexibility and larger vibrating mass. The dowelled plate connection is associated with lower limit PGA for the 6-storey building, but provides limit PGAs in line or even higher than those obtained with the use the stiffer connections for taller buildings. Despite the building lacks from a concrete topping, the diaphragm effect, relying on the connection devices only, was found to be effective although not perfect. The limit PGA values obtained are associated, with reference to the Italian territory, to medium-high seismic areas even for elastic design (q=1.5) for the 6-storey building, whilst an elastic design would confine the use of the taller structures to low seismic areas. Nevertheless, the use of larger q factors (up to 2 for MDC and up to 3 for HDC according to Eurocode8), justified by the adoption of ductile couplers for the vertical unit joints with slight difference in terms of local reinforcement detailing for the taller structures, where the principal stresses are mostly longitudinal and thus flexural, could make the taller buildings checked for medium seismicity areas. Taller 3D cell buildings could also be employed in medium-high seismicity areas if considering a local enhancement of the structure in a limited number of locations, all distributed at the base of the building.

The linear dynamic analysis herein presented might be extended in the near future to non-linear dynamic, in order to assess in detail the performance of both 3D cell reinforced concrete units and their connections.

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