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SUSTAINABLE RESTORATION OF POST-WWII EUROPEAN REINFORCED CONCRETE BUILDINGS

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Abstract

A new approach for the sustainable restoration of the European RC buildings built after World-War II (about 50% of the existing building stock) is proposed in this paper. An additional exoskeleton targeting architectural restyling, energy efficiency upgrade, and structural and seismic upgrading measures is studied for a holistic renovation of the building stock. A new design is introduced to pursue the targets of sustainability and resilience of the intervention. The solution is carried out from the outside, with reduced impairment of the inhabitants and possible building downtime.

Two different structural schemes are investigated: a traditional “Shear Wall” and an innovative “Shell” Solution. In the former, additional shear walls ensure the structural safety, and the new envelope upgrades the energy efficiency. On the contrary, shell solution exploits the shape and the extension of the façade to reduce the dimensions of the structural components and force a new box-structural behaviour. Stresses are reduced to such an extent as to allow for dual use, both energy and structural, of the envelope components. This way, the envelope investigated herein may integrate the energy upgrade technologies and the structural safety systems, requiring a high level of innovation.

In the paper focus is made on the sole structural renovation. The effectiveness of the solution is verified for a reference building, in which the engineered external shell is applied to a typical residential building as an alternative to the shear wall solution.

Keywords: sustainable restoration; double skin; shell structure

1. Introduction: need for a new sustainable and holistic design approach

In recent years, the low-carbon economy challenge has set new goals for the construction sector. European buildings should cut their greenhouse gas emissions respect to the 1990 condition by 20-30% by 2020 and 80-95% by 2050 [1]. The existing building stock is liable for the 35% of energy consumption and 36% of CO_2 emissions in Europe [1]. Moreover, always fewer new buildings are built as a consequence of the economic crisis, making the EU targets unreachable unless a massive energy retrofit intervention is carried out.

Under a structural point of view, 40% of European buildings has already exhausted its nominal structural service life (50 years) and was built before the first seismic regulations, leading to seismic vulnerable structures. This vulnerability is particularly significant considering that recently the seismic hazard in the continent has been further increased. Besides being a safety threat, new researches showed that these poor structural conditions have also a great impact on the environment. Potential building damage or collapse following natural disasters impacts on waste production and CO_2 emissions, greatly affecting the energy savings obtained with a sole energy retrofit intervention [2]. The leading concept of sustainability as an energy-efficiency related issue should thus be updated to include the fundamental requirement structural safety and of resilient society.

In order to achieve the main goal of a low-carbon and resilient building stock, a new sustainable and holistic renovation approach has been recently developed [3-6]. Such approach embraces this new vision by proposing an integrated architectural, energy, and structural upgrading of the buildings and by rethinking the common design approach. Besides improving the impact on the environment by solving all the building deficiencies at the same time, an holistic approach has the further advantages to allow the addition of new living spaces, invest the energy savings to finance the architectural and structural upgrading, reduce the construction site costs and improve its management. On the other hand, the sustainability of the intervention may be guaranteed only by applying a new Life Cycle Design, contemporarily targeting the improvement of the performances and the reduction of the environmental impact along the whole building life cycle. This leads to the definition of new highly-engineered exoskeletons for seismic and energy upgrading and building reshaping, implementing sustainable recyclable materials and dry demountable solutions, which are adaptable to potential new technologies or building functions and easily repairable after earthquakes (Fig.1). This solution is also conceived to be applied from outside, reducing the impairment on the building users and avoiding the costs connected to the interruption or relocation of the internal activities.

This concept, which should be applied to the whole building stock requiring renovation, has been further studied for the upgrade of the post-World-War II RC buildings. These buildings represent about 30-40% of the European existing building stock. They were mainly built to quickly meet the pressing housing demand of those times, often in the absence of any architectural, urban, and environmental general planning, and lacking the main seismic regulations. They are typically clustered in suburbs, featuring obsolete technologies and poor envelope insulation, and resulting in low energy efficiency and living discomfort (Fig.2). For this kind of buildings, possible structural solutions to be implemented into the multi-function exoskeleton were investigated [5, 7]. In particular, ‘shear wall’ or ‘shell’ solutions, both dissipative or non-dissipative, were proposed.

Scope of this paper is the evaluation and comparison of different possible structural non-dissipative exoskeletons, which are designed to improve the seismic response of existing buildings and to be coupled to energy-efficiency upgrading measures. The proposed solutions are firstly introduced, and the main differences between ‘shear wall’ and ‘shell’ retrofit structures are outlined. Alternative technologies are then proposed and applied to a reference building. A simplified design procedure for the estimation of the main structural retrofit parameters is adopted, and the efficiency of the non-dissipative exoskeleton is investigated in terms of enhancement of the existing building seismic response. ‘Shear wall’ and ‘shell’ solutions are then critically commented.

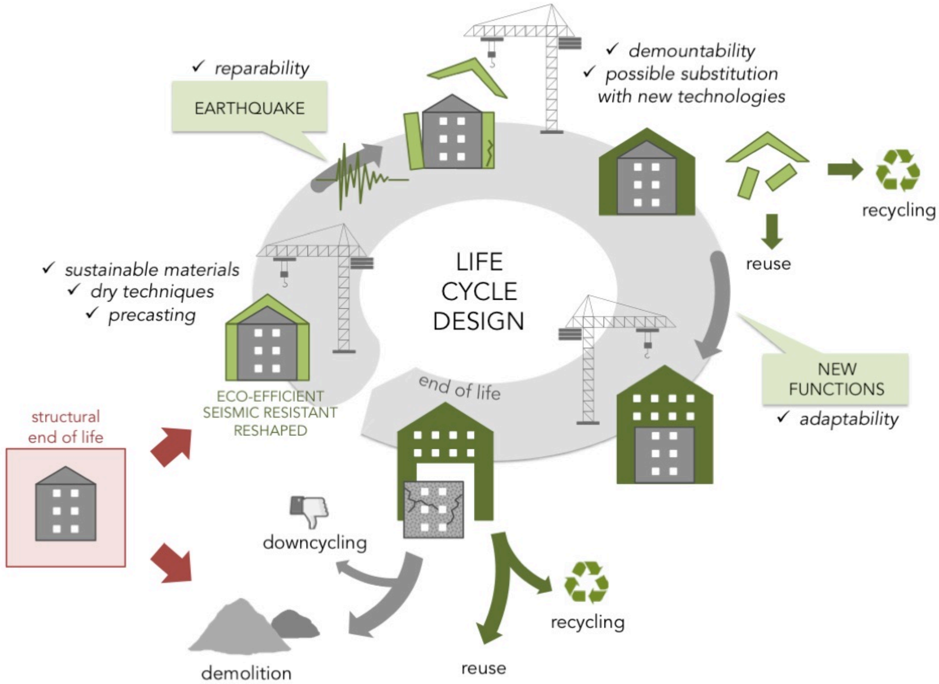

Fig. 1 – Life Cycle Design for sustainability and resilience (from: Marini et al., 2016 [6])

Fig. 2 – Typical post-WWII European RC buildings: an Italian suburb (panoramic view © Microsoft® Bing™ Maps Platform 2016) (left) and building (right)

2. 'Shear Wall' vs. 'Shell' structural solutions

The seismic upgrading of the existing building may be achieved by implementing two alternative retrofit solutions (Fig.3): (i) by complementing the encasing exoskeleton with shear walls, or (ii) more innovatively, by conceiving and exploiting the new façade shell behaviour. These solutions may be easily included within the exoskeleton designed to improve the architectural layout and the energy efficiency, arriving to share the same spaces, the same materials, and the same technologies in the most integrated solutions.

In the 'shear wall' solution, structural safety is entirely entrusted to the external shear walls, whereas the energy efficiency upgrading is guaranteed by an additional thermal insulating layer that encases the walls. Structural elements are part of the exoskeleton on which the energy devices are installed, and the two different systems (energy and structure) work in parallel. The addition of over-resistant stiff walls is a traditional retrofit method for the structural upgrading (Fig.4, left). Usually, different shear wall technologies can be adopted: reinforced concrete walls, steel braced frames, or steel plate shear walls. However, lumping the additional structural strength and stiffness into few elements may result in a significant number of walls to be integrated in the exoskeleton and in high seismic actions to be transferred to the foundation system. This in turn leads to “heavy structures”, and thicker exoskeleton components, especially when stiff masonry infills and light reinforced staircase walls are present. Aimed at reducing the seismic loads into the façade and at the foundation, the new ‘shell’ solution was thus proposed inspired by the box structures [8].

In the ‘shell’ solution, the shape and the extension of the new façade are exploited to reduce the cross section area of each single structural component, resulting in a reduced overload of the foundations and in thinner exoskeleton components. Given the reduced stress demand, the twofold use of thermo-insulating panels as seismic resistant elements can be envisioned, and the new skin becomes both a thermal insulating shell and an in-plane seismic resisting structure. As an example of shell exoskeleton technologies, braced frames or diagrids, usually adopted in high-rise buildings, may be considered (Fig.4, right).

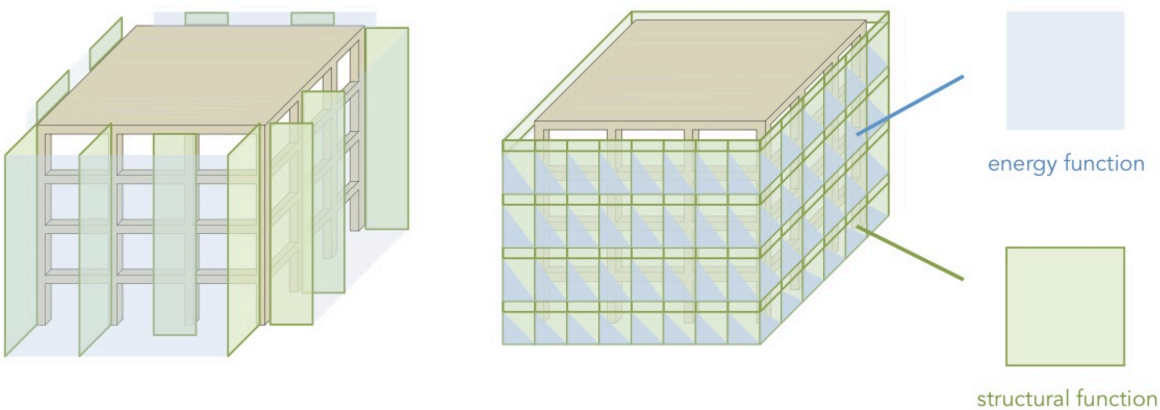

Fig. 3 – Two different solutions that may be implemented into the engineered exoskeleton for the seismic upgrade of existing buildings: shear wall solution (left) and shell solution (right) (from: Marini et al., 2015 [7])

Fig. 4 – Additional shear walls built outside the perimeter of the existing building (left) (from: http://crf.sandia.gov/crf-seismic-retrofit-complete/); examples of shell structures, which could be further conceived to serve as anti-seismic retrofit structures (right): steel shell (Capital Gate, Abu Dhabi), reinforced concrete shell (O-14 Dubai Tower).

Both the ‘shear walls’ and the ‘shell’ structures can be either in adhesion or as an enlargement of the existing external walls, thus enabling maximum flexibility in the architectural restyling and in the energy upgrading solution. In particular, as for the integration between the structural and the energy retrofits, many solutions could be applied: from the curtain wall façades and the solar greenhouses, which can host the new walls in their thickness, to the external thermal insulation cladding, in case of walls located perpendicularly to the existing façade.

3. Design of over-resistant non-dissipative 'shear wall' and 'shell' solutions and application to a reference post-WWII RC building

Both 'shear wall' and 'shell' solutions may be conceived as dissipative or non-dissipative. Dissipative solutions control the seismic response of the existing building by dissipating seismic energy into new devices. Non-dissipative solutions, on the contrary, meet the required targets by adding very stiff and over-resistant external elements, which limit the displacements of the existing structure and withstand the whole seismic action.

It is worth noting that dissipative solutions often implement displacement-activated devices to dissipate energy, thus needing to activate a certain ductility of the existing building. However, when stiff masonry infills, partition walls, or staircase wells are included in the structure, the existing building act as a very stiff structure, which may collapse at low displacements. In all these cases, the dissipative solutions may not be considered the best retrofit options for the seismic upgrade, unless preliminary interventions are applied to increase the ductility of the existing building. As for non-dissipative solutions, attention should be paid to the base shear and the floor in-plane overload [5].

The sole non-dissipative solutions are considered in the following. A procedure is proposed for the estimation of the additional stiffness of the retrofit intervention, and a preliminary design of the new structural elements is proposed for both the 'shear wall' and 'shell' solutions. The design procedure is then applied to a reference building, and nonlinear time-history analyses are performed in order to show the effectiveness of the intervention and the equivalence of the proposed technology options. Finally, 'shear wall' and 'shell' solutions are compared in terms of feasibility and sustainability.

3.1 Structural design of the non-dissipative solutions

Aimed at estimating the additional stiffness of the retrofit system, a 4-step design procedure is presented (adapted from [9]): 1) definition of performance targets; 2) MDOF to SDOF transformation of the existing structure; 3) estimation of the additional stiffness of the retrofit intervention; 4) SDOF to MDOF transformation of the retrofitted structure and design of new structural elements.

In the first step of the design, the behaviour of the building in the as-is situation is estimated and the performances of the building after the retrofit intervention is defined. The aim of non-dissipative interventions is to maintain the building into the elastic field, which is particularly useful for brittle structures. When no extensive damage to the existing infill walls and staircase walls are accepted, target displacements of the retrofitted building are usually very low. High additional stiffness is thus required, leading to very high base shear and floor in-plane overload. A maximum target base shear should thus be defined, based on the maximum shear that may be taken by the foundations of the additional elements. Finally, the in-plane capacity of the existing floors should be estimated and compared to the resulting demand; when the floor in-plane demand is higher than its capacity, a retrofit of the floor diaphragms should also be considered.

Once the final targets are selected, the existing structure is transformed from MDOF to SDOF and the resulting capacity curve is bilinearized, following standard bilinearization procedures.

In the third step, the stiffness of the retrofit intervention, which is the only design parameter, is determined by considering a global SDOF system with the equivalent mass of the building and stiffness equal to the sum of the stiffnesses (frame and retrofit intervention) (Fig.6, right). The period corresponding to the target displacement of the new SDOF system is obtained from the displacement response spectrum, and the total stiffness of the system is calculated according to:

$$K_{tot} = m^* \left(\frac{2\pi}{T}\right)^2 \tag{1}$$

Since the behaviour of the structure is elastic, the stiffness of the retrofit intervention ($K_{retrofit}$) is the difference between the total (K_{tot}) and the initial stiffness (K_{frame}):

$$K_{retrofit} = K_{tot} - K_{frame} \tag{2}$$

Once the additional stiffness required to the SDOF system is determined, the geometry of the new elements could be directly computed. The cross section may be constant along the height of the building, or may vary in order to take into consideration the first mode shape of the structure (a simplified procedure may be found in [9]). A shear wall or shell solution may thus be implemented into the structural exoskeleton.

Regarding the shear wall solution, elastic walls connected with rigid links may be adopted. **Shear elastic walls** may be designed considering the additional stiffness equally distributed among the walls and along the building height. The dissipative walls are treated as cantilevers of height $l*$ with a lumped load at the top; assuming a base b_w of the wall, the width h_w may thus be calculated as:

$$h_w = \sqrt[3]{\frac{12\ l^{*3}\ K_{retrofit}}{3Eb_w}} \tag{3}$$

where E is the elastic modulus and $l*$ is considered in the following equal to *3/4* the height of the building.

When steel braced frames are adopted, the design of the elements should lead to walls of equivalent stiffness by applying the following equations:

$$F = 2F_d cos\alpha = 2K_d d_h cos^2\alpha \tag{4}$$

$$K_d = \frac{E_d A_d}{d} = \frac{E_d A_d}{h} sin\alpha \tag{5}$$

$$K_h = 2\frac{E_d A_d}{h} sin\alpha cos^2\alpha \tag{6}$$

where α is the inclination of the diagonals, F and F_d are the horizontal and diagonal forces, d_h is the horizontal displacement, K_h and K_d are the horizontal and diagonal stiffnesses, E_d, A_d, and d are the elastic modulus, the area, and the length of the diagonals, and h is the height of the braced frame modulus. However, when the existing building is a stiff infilled frame, the additional elastic walls and the required foundations may reach significant dimensions. A shell solution may thus be required such as in the box structures [8]. Among the existing structural technologies, stiff braced frames or diagrids can be adopted as shell exoskeletons.

3.2 Existing reference building

The design procedure for non-dissipative systems is applied for the seismic upgrade of a reference building. An Italian building typical of the post-WWII European stock is considered. The building has rectangular plan, three floors, and a basement. Three longitudinal frames and two transversal frames at the ends constitute the building structure. Floors are one-way lightweight RC ribbed slabs, and closures are two-brick-leaf masonry infills along the whole perimeter (12+8 cm). The embodied carbon associated to the seismic risk of the considered building [2] is about 87% of the annual operational carbon if thermal refurbishment is carried out without seismic retrofit, while passing to 10% whether structural upgrading is included.

The non-dissipative solutions are designed considering the existing building in its as-is situation, i.e. with stiff masonry infills and lightly reinforced staircase walls not designed for horizontal loads. The building is modelled as a tridimensional structure, adopting beam elements with lumped plasticity according to Takeda hysteresis rule [12]. The infill walls are modelled with single struts converging in the frame nodes [11] and the floors act as rigid diaphragms. The columns are considered fixed at the base. The first level of the staircase walls is not considered participating to the seismic resistance due to their downgrading by means of few vertical cuts [13] [14]. The existing building, the geometry of the external transverse frame and the building plan are reported in Fig.5.

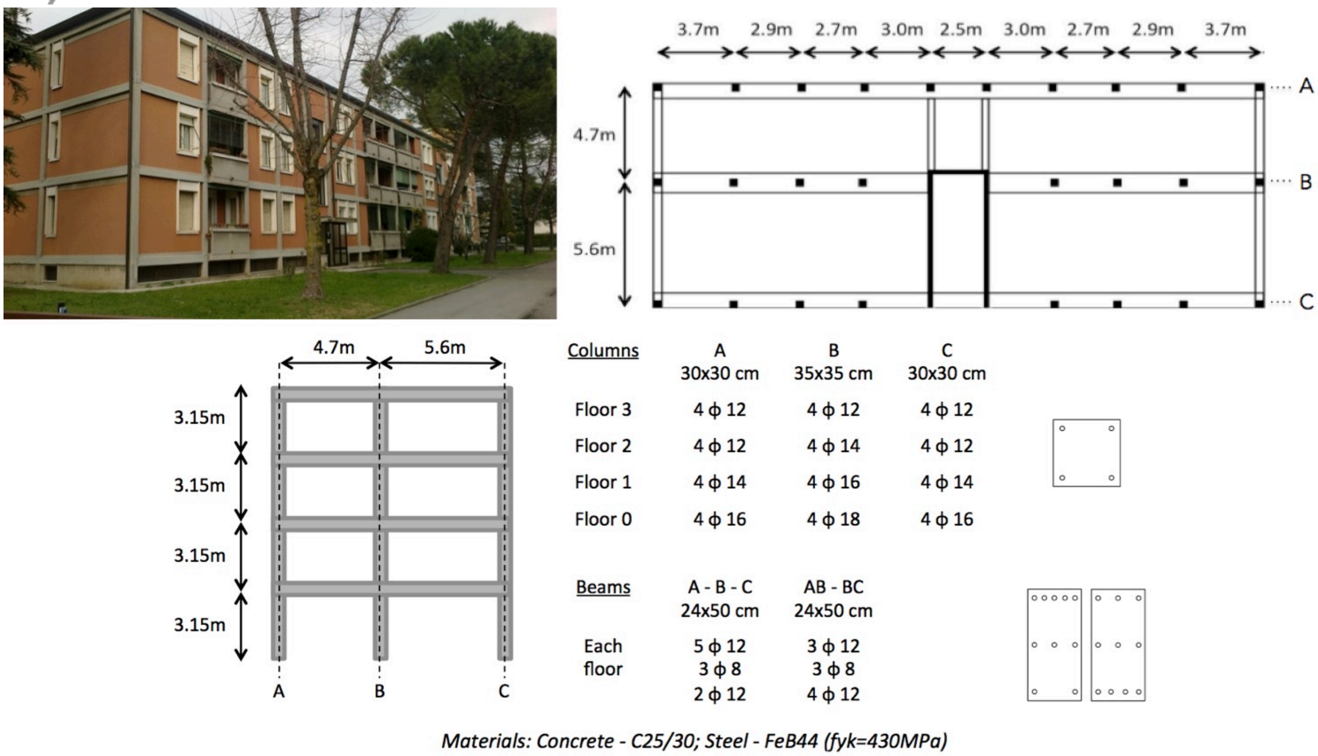

Fig. 5 –Reference Italian building built in 1972 representative of the European post-WWII RC building stock (top, left); plan view (top, right) and external transverse frame (bottom) of the 3D finite element model.

For the definition of the Performance Targets the Life Safety Limit State is selected as Performance Level. In this case, since the structure may be severely damaged for very low values of drift because of its interaction with the infill walls, a target drift of 0.1% is imposed in order to maintain the structure into the elastic range. The shear at the base of the existing building should be reduced after the retrofit intervention, while a maximum shear of 250-300kN/m is considered admissible at the foundation of the new elements. Finally, the floor in-plane load demand is checked and compared with capacity in order to ensure diaphragm behaviour. In this case, supposing the activation of a tied arch into the diaphragm, a maximum shear capacity equal to 625kN is considered for the ‘shear wall’ solution and equal to 1250 kN for the ‘shell’ solution [15, 5].

The existing structure is then transformed from MDOF to SDOF. The behaviour of the structure is considered as linear up to a displacement of about 1 cm and a base shear of about 600 kN; the resulting stiffness of the existing infilled frame is thus equal to 60 kN/mm. The equivalent SDOF is defined by the following characteristics: T=0.63s, d_{el}=10mm, K_{fr}=60kN/mm. The bilinearization of the curve is shown in the left side of Fig.6.

Once the target displacement is selected and the equivalent SDOF system defined, the correspondent fundamental period of the retrofitted structure is obtained from the displacement spectrum. Given the participating mass of the first mode of the building, the correspondent global stiffness of the retrofitted building is determined, and the stiffness of the retrofit intervention is determined as a difference between the total stiffness and the stiffness of the initial structure (Fig.6, right):

$$T(S_d) = 0.185\ s \tag{7}$$

$$K_{tot} = m^* \left(\frac{2\pi}{T}\right)^2 = 0.594\left(\frac{2\pi}{0.185}\right)^2 = 685\ kN/mm \tag{8}$$

$$K_b = K_{tot} - K_{fr} = 685 - 60 = 625\ kN/mm \tag{9}$$

A correction factor equal to 1.3 is applied in order to account for uncertainties related to ground motion variability and influence of higher modes effects; the calibration of this parameter is subject of ongoing research a stiffness equal to 212.25 kN/mm is thus considered for each one of the four walls.

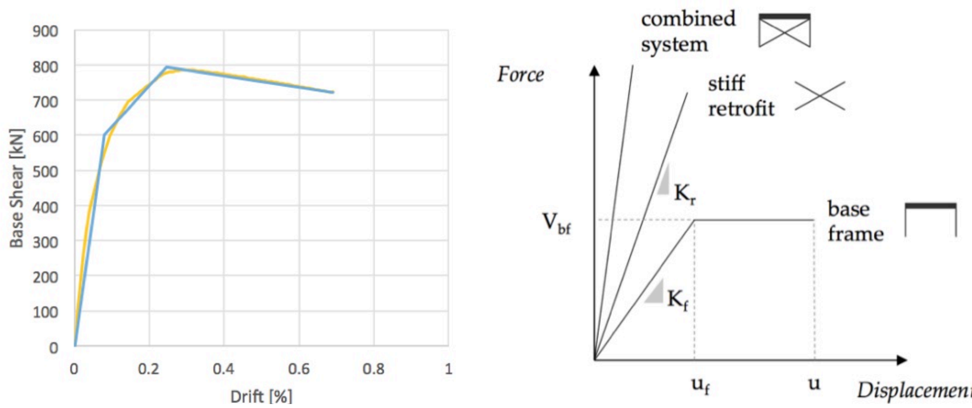

Fig. 6 – MDOF to SDOF transformation of the existing building capacity curve (left), and combination of this base curve with the linear representation of the elastic retrofit intervention (right).

The resulting additional stiffness of the retrofit intervention is finally considered in order to design the new elements. As for the 'shear wall' solutions, four steel walls with 2.5m depth made by tubular elements (A=100200mm^2) are considered. However, in order to reduce the dimension of the elements while maintaining the same results, the same stiffness may be obtained by exploiting the whole surface of the façade. A braced frame and two types of diagrid solutions are proposed. In diagrid 'A' the grid height of 1/3 of the floor height while in diagrid 'B' the grid and floor height coincide. In order to have the same target displacement, different tubular profiles are considered in different 'shell' solutions: D=273.5mm and s=25mm in the braced frame; D=114.3mm and s=10mm in diagrid 'A'; D=219.1mm and s=16mm in diagrid 'B'. For the sake of simplicity, the same dimensions are considered at each floor. Commercial steel tubular profiles are adopted, thus leading to slight variations of the building seismic response.

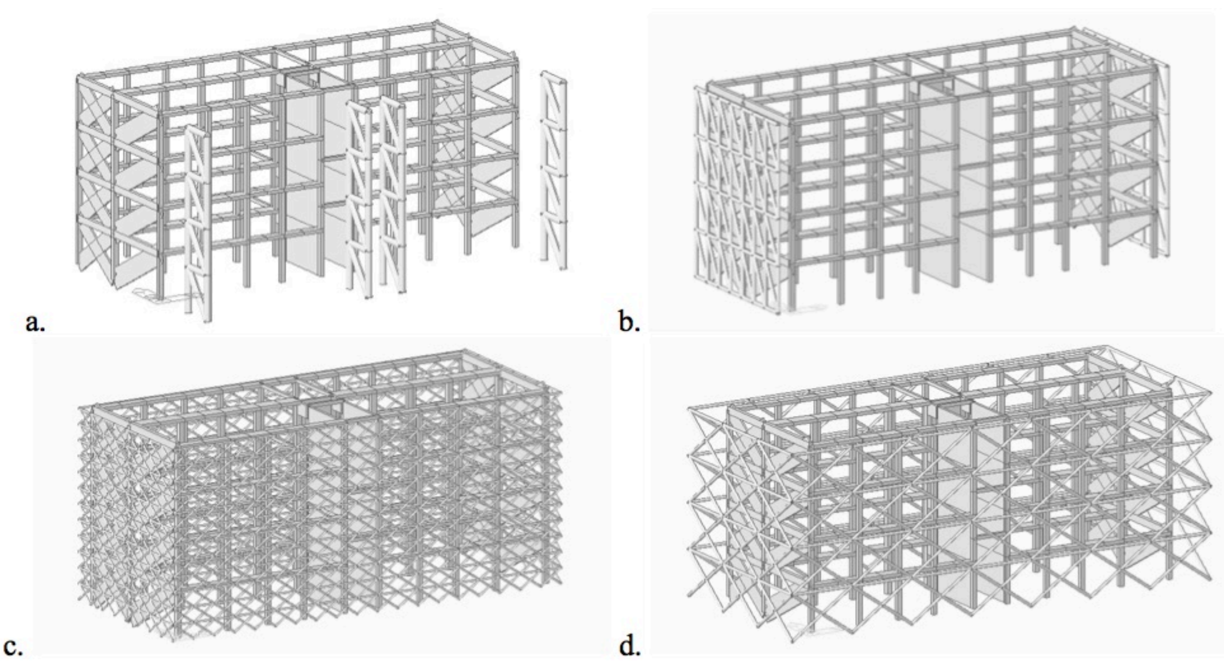

Fig. 7 –Non-dissipative solutions for the retrofit of the reference existing building: 'shear wall' solution – elastic shear walls (a); and 'shell' solutions – braced frame façade (b); diagrid A (c); diagrid B (d).

3.3 Discussion of the results: ‘shear wall’ vs. ‘shell’ structures

The proposed solutions are evaluated by comparing the results of the time history analyses performed on the existing building prior and after retrofit adopting the software MidasGEN v.2012 [16]. Supposing the building is located in L’Aquila (one of the city with the highest site seismicity in Italy), a set of seven records compatible with the life safety limit state spectrum is selected [17]. The equivalence of the different solutions, designed to reach the same target displacement, is firstly evaluated. The effectiveness of the ‘shear wall’ and ‘shell’ solutions in withstanding the seismic loads is then discussed. Finally, sustainability issues are briefly addressed.

The effectiveness of the non-dissipative structural retrofit solutions is evaluated based on the achievement of the target global drift. Some other important building responses are then controlled to define the structural feasibility of the intervention and the best retrofit option. In particular, the resulting base shear is compared to the maximum shear flow allowable by the foundation of the new structural elements. Then, interstorey drift and storey shear are estimated aimed at evaluating the damage on structural elements and on drift-sensitive nonstructural elements, such as infill and partition walls and estimating the stresses into the existing floor diaphragms. When all these parameters are taken under control by the retrofit intervention, the building is considered as safe for the human life and the damage is reduced assuring low repair costs and short building downtime after the seismic event.

The results in terms of total and interstorey drift, residual drift, and base and storey shear are shown in the following. Since the existing building in the as-is situation collapses under the effect of each selected earthquake due to its high stiffness and to a soft storey mechanism triggered by the brittle behaviour of the infills, the results are compared with the selected targets.

As regards the total drift, it is verified that the elements of the retrofitted frame remain elastic and that the infill walls do not reach their ultimate capacity – a little damage of the infills is accepted since a Life Safety Limit State is considered. Being the total roof drift the main target in input to the design procedure, it may be noted from Fig.8 (top) and Fig.9 (left) that all the retrofit solutions lead to drift smaller than 0.1%.

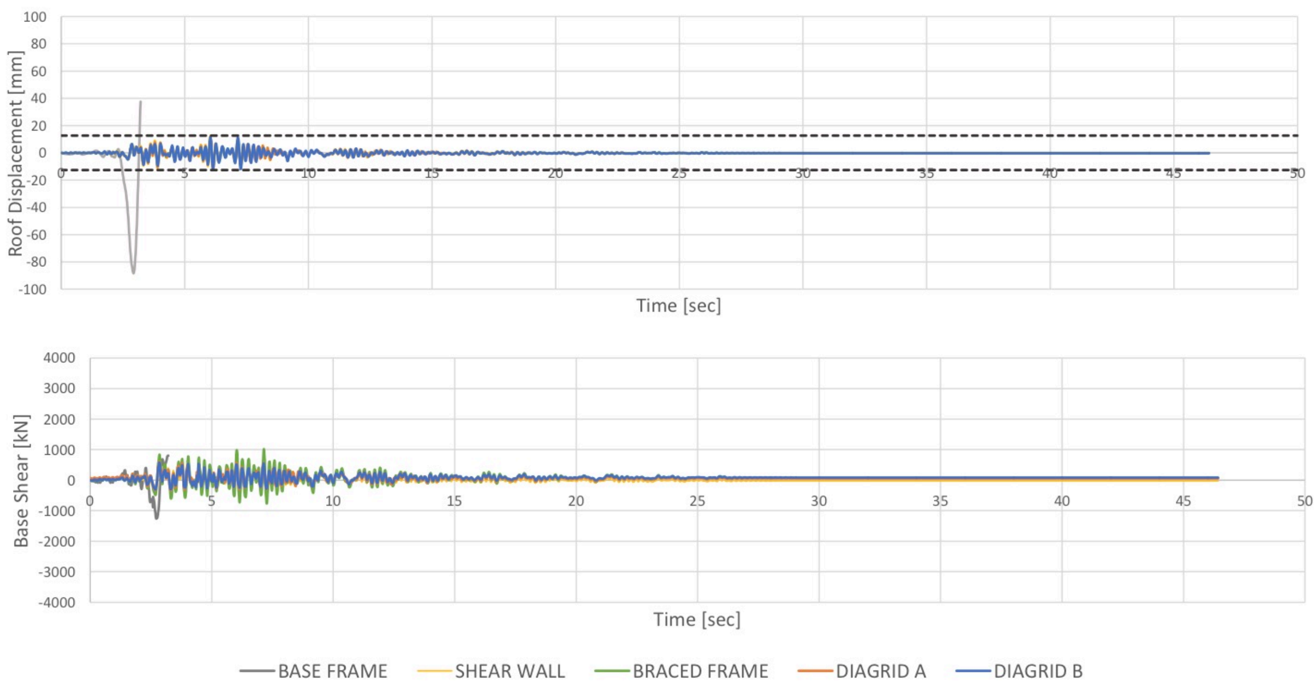

Fig. 8 –Roof displacement (top) and shear at the base of the existing building (bottom) for equivalent solutions for the same earthquake record. The target drift of ±0.01% is reported with the dotted black lines.

As for the base shear, it may be noted that, while the shear at the base of the existing building is reduced thanks to the addition of the new stiff elements, the total base shear is highly increased (Fig.9). This increase should be compared to the maximum capacity of the new foundations (a target of 250-300 kN/m is considered). It is worth noting that under this point of view, the shell solution is better than the shear wall solution since the exploitation of the extension of the façade allow a distributed foundation. On the contrary, in the shear wall solution, the foundation should be lumped at the wall base, leading to higher load concentration. In this case, the high loads at each wall (V=1651 kN; M=2188 kNm) cannot be supported by shallow foundation systems, and the solution may not be accepted. All the solutions, having the same stiffness, present similar base shear time histories (Fig.8, bottom); however, braced frame solutions, being slightly stiffer, lead to higher shear at the base of the frame, particularly at the staircase walls, and total base shear (Fig.9, center and right).

Regarding interstorey drift and storey shear, similar results are obtained for the different solutions. The interstorey drift distribution achieves the target of 0.1% at each floor (Fig.10, left). The storey shear is highly increased by the non-dissipative solutions due to the high increase of the global stiffness of the system. In Fig.10 (right), the average storey shear of each solution is compared to the maximum capacity of the building floor diaphragms, which are different for ‘shear wall’ and ‘shell’ solutions, as previously introduced. In any case, strengthening of the existing floors at each level of the building is required. It should be noted that the results are expressed as the average values of the time history response at each floor, so representing the envelope of the average responses at different instants.

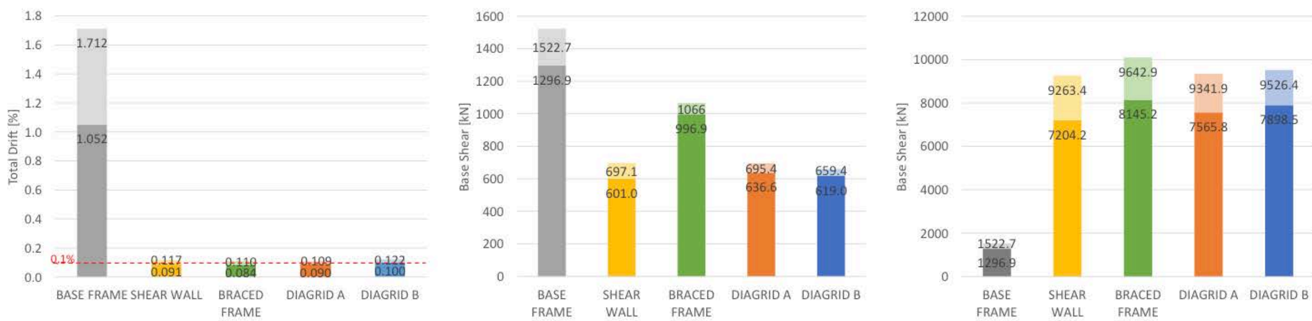

Fig. 9 –Average and maximum roof displacement (left), shear at the base of the existing building (center), and total base shear (right) for equivalent non-dissipative shell solutions.

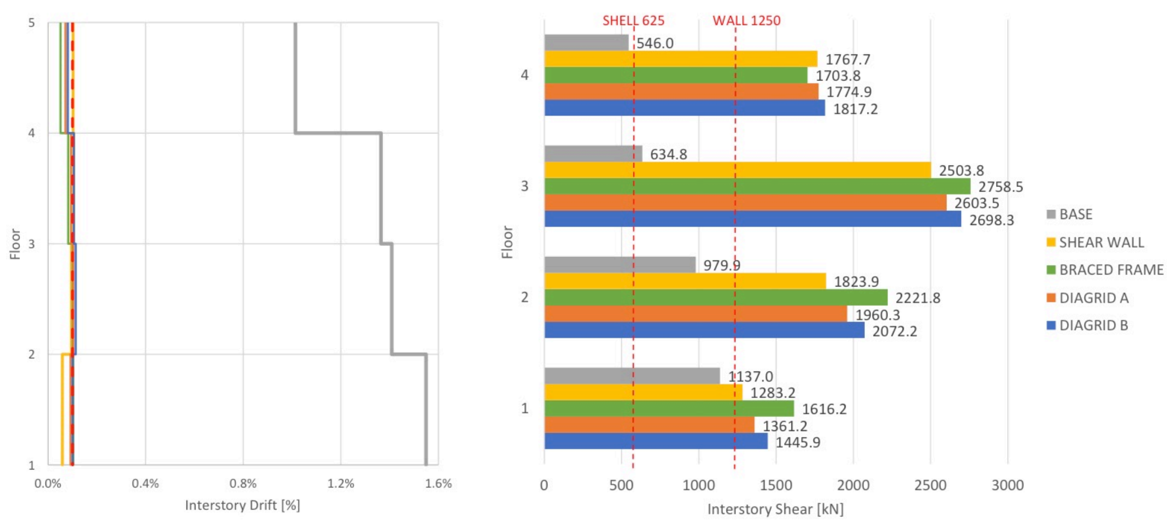

Fig. 10 –Average interstorey drift along the building height for different solutions – the dotted red line represents the 0.1% interstorey drift target (left); average floor shear along the building height for different solutions; the dotted black line represents the limit capacity of the floor diaphragms. The graphs represent the envelope of the building response; the values at each floor may be reached at different moments.

4. Conclusions

Targeting sustainability of the built environment, previous studies have shown how coupling seismic retrofit to energy upgrading reduce the impact on the environment since it reduces the potential risk of building damage or collapse after earthquakes, showing the importance of applying the principles of the sustainable design to retrofit solutions in order to guarantee an eco-efficient and resilient building stock. In this paper, a holistic and sustainable retrofit solution is proposed and applied to a reference post WWII European building.

Targeting sustainability, fully demountable steel dry solutions are proposed in order to guarantee easily reparability after natural disasters and full adaptability to new technologies and new functions. Differently to cast-in place solutions or composite materials, the proposed technologies guarantee total recyclability and/or reuse of the retrofit intervention at the end of life of the existing RC structure. Non-dissipative structural solutions are considered herein adopting two different configurations: ‘shear wall’ or ‘shell’. A 4-step simplified performance-based design procedure is proposed, which can be adopted for the preliminary design of each of the investigated solutions. This procedure has been applied to a reference building, and the solutions are compared by means of time history analyses.

Both ‘shear wall’ and ‘shell’ solutions are efficient in the control of the seismic response; however, ‘shell’ solutions may be considered more efficient and sustainable than ‘shear wall’ solutions. Under a structural point of view, ‘shell’ solutions entail distributed foundations, thus allowing higher shear flows at the base. On the contrary, lumping the intervention into shear walls imply lumped foundations which might require deep foundations, as for instance micropiles. In addition, while ‘shear wall’ solutions imply lower in-plane loads into the floor diaphragms, both ‘shear wall’ and ‘shell’ solutions might require the retrofit of the floor diaphragms if high in-plane loads are generated by the addition of the stiff exoskeleton. Regarding the sustainability of the interventions, both the solutions may be conceived to be fully demountable, recyclable, and adaptable.

5. References

[1] Marini A, Passoni C, Riva P, Negro P, Romano E, Taucer F (2014): *Technology options for earthquake resistant, eco-efficient buildings in Europe: Research needs, Report EUR 26497 EN. JRC87425*. ISBN 978-92-79-35424-3. doi:10.2788/68902. Publications Office of the European Union.

[2] Belleri A, Marini A (2016): “Does seismic risk affect the environmental impact of existing buildings?” *Energy and Buildings*. 110, 149-158. doi:10.1016/j.enbuild.2015.10.048.

[3] Feroldi F, Marini A, Belleri A, Passoni C, Riva P, Preti M, Giuriani E, Plizzari G (2014): “Sustainable seismic retrofit of modern RC buildings through an integrated structural, energetic and architectural approach adopting external engineered double skin façades.” *Progettazione Sismica*, 5 (2), pp. 1-15 (in Italian).

[4] Marini A, Belleri A, Feroldi F, Passoni C, Preti M, Riva P, Giuriani E, Plizzari G, (2015): “Coupling energy refurbishment with structural strengthening in retrofit interventions.” *SAFESUST Workshop, 26-27 November 2015*, Ispra, Italy.

[5] Passoni C (2016): *Holistic renovation of existing RC buildings: a framework for possible integrated structural interventions*. Ph.D. thesis, University of Brescia.

[6] Marini A, Belleri A, Passoni C, Feroldi F, Preti M, Metelli G, Giuriani E, Riva P, Plizzari G (2016): “Integrated architectural, energy and seismic renovation of the reinforced concrete building stock targeting sustainability and resilience.” Submitted for possible publication to European Journal of Environmental and Civil Engineering.

[7] Marini A, Passoni C, Belleri A, Feroldi F, Preti M, Riva P, Giuriani E, Plizzari G (2015): Why coupling energy refurbishment with structural strengthening interventions, *Italian National Project PRIN, Final Report*, Coordinator: Montuori M. University of Brescia.

[8] Giuriani E, Marini A (2008): “Wooden roof box structure for the anti-seismic strengthening of historic buildings.” *Journal of Architectural Heritage*, 2 (3), pp. 226-246.

[9] Guo JWW, Christopoulos C (2013): “Performance spectra based method for the seismic design of structures equipped with passive supplemental damping systems.” *Earthquake Engineering and Structural Dynamics*, 42, pp. 935-952.

[10] Moon KS, Connor JJ, Fernandez, JE (2007): "Diagrid Structural Systems for Tall Buildings: Characteristics and Methodology for Preliminary Design." *The Structural Design of Tall and Special Buildings*, 16, pp. 205-230.

[11] Decanini LD, Gavarini C, Bertoldi, SH (1993): "Telai tamponati soggetti ad azioni sismiche, un modello semplificato: confronto sperimentale e numerico." Acts of the VI Convegno Nazionale di Ingegneria Sismica in Italia, Perugia, 1993. (in Italian)

[12] Otani S (1974): SAKE: A Computer Program for Inelastic Response of R/C Frames to Earthquakes. Civil Engineering Studies, University of Illinois at Urbana, Champaign, Report SRS-413.

[13] Preti M, Bettini N, Plizzari GA (2012): "Infill walls with sliding joints to limit infill- frame seismic interaction: large-scale experimental test", *Journal of Earthquake Engineering*, 16, pp. 125-141.

[14] Belleri A, Passoni C, Marini A, Riva P (2016): "Hinged-wall solutions for the structural strengthening of existing RC buildings." *Acts of the fib Symposium 2016*, 21-23 November 2016, Cape Town, South Africa.

[15] Feroldi F (2014): *Sustainable renewal of the post WWII building stock through engineered double skin, allowing for structural retrofit, energy efficiency upgrade, architectural restyling and urban regeneration*. Ph.D. thesis, University of Brescia (in Italian).

[16] MidasGEN (2012): *Analysis Manual for MidasGEN.*

[17] Iervolino I, Galasso C, Cosenza E, REXEL (2010): computer aided record selection for code-based seismic structural analysis. *Bulletin of Earthquake Engineering*, 8, pp. 339-362.