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Seismic strengthening through external exoskeleton

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Highlights

Within the framework of the European project Pro-GET-onE, two cases of reinforcement obtained by applying a steel exoskeleton connected to the existing structures will be presented. The first case refers to a reinforced concrete building where a three-dimensional steel addition is considered with the aim to provide extra-space with high energy performance envelopes and structural improvement to the existing structure. The second case concerns a very common type of residence in the Netherlands, the terraced house. In this case the intervention will focus on the realization of a planar frame leaning against the existing wall weakened by the openings. In both cases this strengthening strategy gives an added value to the existing building with the integration of different technologies to achieve a multi-benefit approach by a closer integration between different aspects such as social, safety and energy and that is the reason that leads to this choice of intervention instead of the traditional ones.

Abstract

The seismic hazard in Europe is one of the most critical issues of civil engineering. The necessity of improving existing buildings, in terms of energy and structure is always a new challenge for designers. The use of integrated improvement systems can be the solution to common obstacle from the project to the realization, such as the invasiveness, the cost and the duration of the construction phase. The current scenary is rich in different intervention techniques due to the heterogeneity of the buildings. The study focuses on two cases of seismic reinforcement through the use of steel exoskeletons in different contexts through different design solutions. Following the description of the issues related to the vulnerability of the two case studies, the procedures for evaluating the improvement are illustrated. Finally, the results deriving from the application of the strengthening structures are presented, showing ample margins for improvement in both cases up to the achievement of demand values.

Keywords

Steel exoskeleton, Seismic retrofit, Integrated structural and energetic retrofit, Modern R.C. buildings, Masonry terraced houses

1. INTRODUCTION

The EU research project Pro-GET-onE focuses on the identification of integrated technological solutions that allow improvements in the energy performance and seismic safety of existing residential buildings. The criteria to follow are those of the search for sustainable solutions and fast realization,

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which do not involve the interruption in the use of buildings and which also offer functional improvements to residential units, with the extension of living space, as intrudeced by A. Ferrante et al. [1]. The research study is based on the unprecedented integration of different technologies to achieve a multi-benefit approach by a closer integration between different aspects:

1. *energy requirements* – by adding (or substituting the existing with) new prefab and plug and play high energy performing envelopes and HVAC (Heating, Ventilation, Air Conditioning) systems;
2. *safety* – using appropriate external structures to increase the overall structural capacity of the building, while supporting the new envelope consisting in timber-based components for opaque parts/surfaces, and aluminum, glass, PV Photovoltaic, solar panels;
3. *social and economic sustainability* – increasing the real estate value of the buildings and the desirability of retrofit options by providing tailored and customized solutions for users, owners and house managers, increasing safeness and minimize disturbance of inhabitants.

Information from SHARE Project [2] indicates Italy, Greece, Romania and the Mediterranean countries of the European Union as the areas with the highest probability of natural earthquakes. In these areas, recent seismic events have shown how relevant is the issue of seismic vulnerability for existing buildings of reinforced concrete since many of these were designed without any reference to anti-seismic criteria. A different case concerns the Dutch province of Groningen. In this area, as will be shown in the fourth paragraph, the seismic action is of an induced nature and is caused by gas extraction carried out in the last decades. The traditional masonry houses have never been designed for seismic actions, as natural earthquakes do not occur in this region.

In the design process of a seismic improvement intervention, after a careful assessment of the vulnerability of a building, it is necessary to proceed with the choice and the adoption of streghtening interventions able to allow the structure to support the horizontal action of the seismic zone. This choice depends on numerous factors, including invasiveness, cost, global behavior and critical aspects of the structure. The Pro-GET-onE strategy proposes a type of integrated seismic improvement intervention that excludes the displacement of the inhabitants and at the same time entails an energy improvement through a system of volumetric additions in one case or with a planar addiction in the other. These objectives can be achieved thanks to the positioning of the new reinforcing structures outside the existing building, through the use of steel exoskeletons; technique, to date, in experimentation.

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Projects in which this strategy was used are the requalification of the office and warehouse buildings of the Magneti Marelli factory in Crevalcore (Italy) made by Teleios Srl [3, 4], and the seismic reinforcement of the complex of the Department of Engineering Rural and Topographic of AUTH, located in Thessaloniki, Greece [5]. However, in the cases described, the exoskeleton does not provide integrated solutions for energy improvement and the possible volumetric expansion, as in the presented case. In the case of Groningen many houses have already been strengthened, including upgrading to Zero Energy standards.

2. VULNERABILITY ASSESSMENT METHODS

In front of complex and articulated buildings, the tendency is to make the assessment explicit with few equivalent parameters of capacity, such as displacement, acceleration, return period, etc. However, as the number of components allows it, it is possible to accurately represent shortages, reinforcement interventions and improved results on the members subject to horizontal actions due to the earthquake.

European regulations define the procedures to be followed to evaluate existing structures in Eurocode 8 part 3 [6]. In addition to evaluations through modal and dynamic analysis with response spectrum, the complexity of the existing structure, the partial knowledge of their geometric-mechanical characteristics, together with the uncertainties on the seismic input, lead the choice towards analysis methods characterized by intermediate levels of complexity. This is the case of pushover analysis that, while reproducing the salient expectations of the non-linear response, it is based on the assumption of static actions applied to the structure.

3. THREE-DIMENSIONAL EXOSKELETON, ATHENS CASE STUDY

The structural improvement through the application of an external three-dimensional exoskeleton in steel has been evaluated on the student house in Athens, pilot case of the project in the Mediterranean area. The study of the metallic structural system to be adopted in combination with the existing structure in reinforced concrete lead to two possible different strategies: the stiffness or the damping increase. The former provides, by the application of a rigid structure (such as to affect the resistant capacities of reinforced concrete frames), the increasing of the overall capacity of the structure; while the second one provides the construction of a new relatively independent

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external structure that is adequately connected to the existing building. In this solution, the structural scheme provides dampers in strategic positions such as to increase the energy dissipation of the seismic action.
In this section, the analyzes conducted aim to demonstrate the incidence due to the application of the external exoskeleton, using the stiffness increase strategy. The same results are shortly presented in A. Ferrante et al. [7] concerning the hypothesis on the Athens case study.

3.1. DEFINITION OF THE SEISMIC ACTION

Within the scope of EN 1998 [8] the earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, henceforth called an "elastic response spectrum". The horizontal seismic action is described by two orthogonal components assumed as being independent and represented by the same response spectrum. The elastic response spectrum S_e (T) is defined by the following expressions [8]:

$$0 \leq T \leq T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right]$$

$$T_B \leq T \leq T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5$$

$$T_C \leq T \leq T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C}{T}\right]$$

$$T_D \leq T \leq 4s : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C T_D}{T^2}\right]$$

η is a coefficient that takes into account the viscous damping ξ. The values of the periods T_B, T_C and T_D and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type. The spectral shape must be determined on the basis of the identified site, of the ground, of the type of building and of the limit state of interest; in this work the limit states of Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC) are used.

- LS DL – PVR = 63%; TR = 50 years; VR = 50 years - ag/g = 0,067;
- LS SD – PVR = 10%; TR = 475 years; VR = 50 years - ag/g = 0,16;
- LS NC – PVR = 5%; TR = 975 years; VR = 50 years - ag/g = 0,212.

3.2. ANALYSED BUILDINGS AND STRUCTURAL MODELING

The case study represents a part of the entire building of the student house, it is divided with a seismic joint from the rest and it is considered isolated (Figures 1 and 2).

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Based on this scheme, the necessary information for the definition of the model (materials characteristics, reinforcements and loads) was assumed considering the recurrent characteristics of traditional constructions made in Greece with reinforced concrete structure of the '70s/'80s. The existing structure consists of five floors of 2,80m height each, including flat roof, with a total plan size of 22,30x12m. The horizontal structures are made of concrete slabs that can be considered as diaphragm constraints. The C20/25 (reference value, $f_{ck,\ cyl}$ = 20Mpa) concrete class was used and the FeB32k (reference value, $f_{yk} \geq 315$ MPa) smooth bar with low performance for reinforcement. The vertical elements are piers of 1,20x0,25m connected by beams in both directions, except for the central span in which the only connecting element between the partitions is the slab.

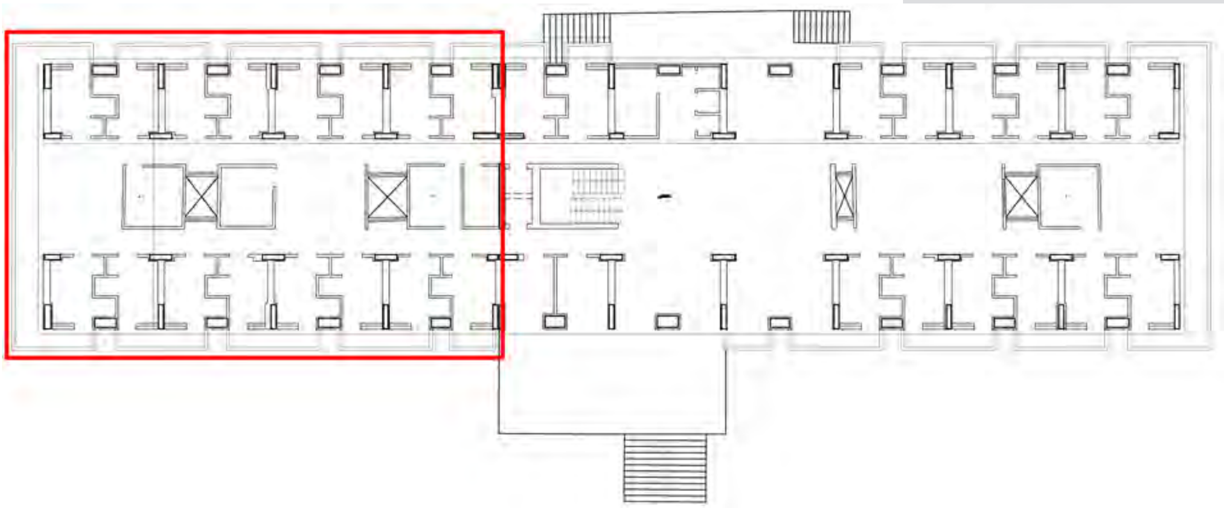

Figure 1. Architectural horizontal cross section of the Athens student house. The part of the structure subject to the improvement intervention is indicated.

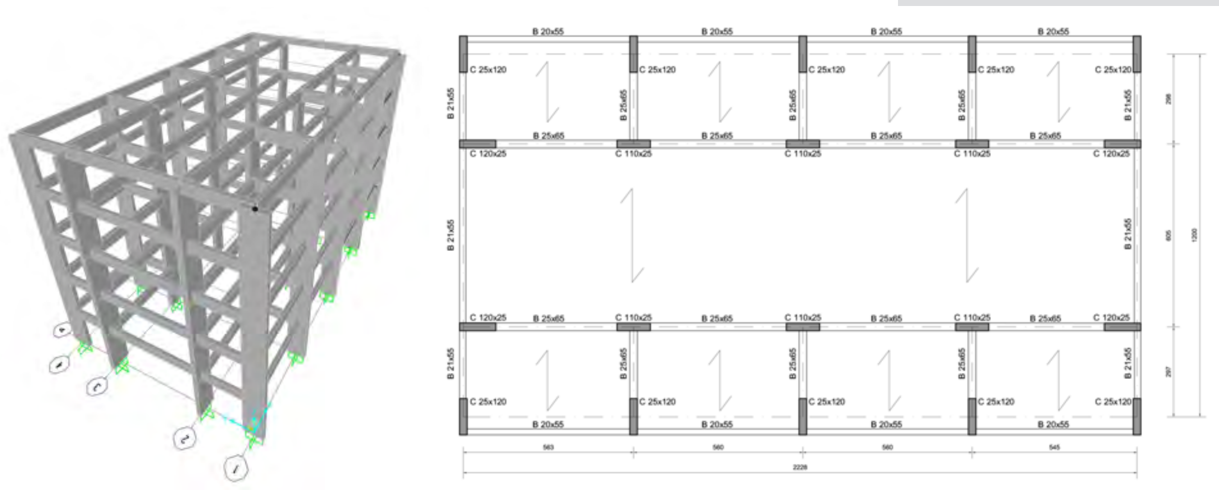

Figure 2. Finite element model (on the left) and structural plan of the type floor (on the right) of the case study based on the Athens student house.

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The reinforcement bars have been defined and imputed by a simulated project using the permissible stress design. Sections, reinforcement and imposed loads are listed below in tables 1 and 2.

Frame	Section (cm)	Reinforcement	
Beam1 – Roof	20x55	$A'_s = 6{,}53\ cm^2$	$A_s = 4{,}52\ cm^2$
Beam1 – n floor	20x55	$A'_s = 10{,}56\ cm^2$	$A_s = 4{,}52\ cm^2$
Beam2 – Roof	25x65	$A'_s = 13{,}35\ cm^2$	$A_s = 4{,}52\ cm^2$
Beam2 – n floor	25x65	$A'_s = 15{,}71\ cm^2$	$A_s = 4{,}52\ cm^2$
Beam3 – Roof	21x55	$A'_s = 4{,}52\ cm^2$	$A_s = 4{,}52\ cm^2$
Beam3 – n floor	21x55	$A'_s = 7{,}67\ cm^2$	$A_s = 4{,}52\ cm^2$
Beam4 – Roof	25x65	$A'_s = 4{,}52\ cm^2$	$A_s = 4{,}52\ cm^2$
Beam4 – n floor	25x65	$A'_s = 4{,}79\ cm^2$	$A_s = 4{,}52\ cm^2$
Column1	25x120	$A_{s,tot} = 16{,}08\ cm^2$	$st.\emptyset 6/250$
Column2	110x25	$A_{s,tot} = 16{,}08\ cm^2$	$st.\emptyset 6/250$

Table 1. Sections and reinforcement of the reinforced concrete profiles.

Floors (Reinforced concrete slab)		
Self-weight	Dead load	Live load
$3{,}75\,\frac{kN}{m^2}$	$n\,floor - 3{,}13\,\frac{kN}{m^2}\ \vert\ Roof - 1{,}56\,\frac{kN}{m^2}$	$C1 - 3\,\frac{kN}{m^2}\ \vert\ H - 0{,}34\,\frac{kN}{m^2}$
External walls (Masonry – 25 cm)		
Self-weight	Dead load	Live load
–	$10\,\frac{kN}{m}$	–

Table 2. Load patterns.

3.3. INITIAL STATE

In the evaluation of the vulnerability of the initial state, modal analysis, linear dynamic analysis with response spectrum and non-linear static analysis were performed, the latter in the two main directions of the building.

The modal analysis shows that the main vibrating mode is that in the transversal direction Y (U2), with an activated mass percentage of 75,1% and a period of 0,536 s. From the dynamic linear analyzes the maximum displacements have been extrapolated which at the limit state of damage correspond at the top to 2 cm in longitudinal directions (X) and 2,6 cm in the transversal (Y). Finally, after defining the control point in the barycentre of the structure's roofing plan, using a distribution of lateral forces proportional to the storey masses, the capacity curves of the structure in the two directions were determined trough the non-linear static analysis. By identifying the steps in which the three limit states are crossed, it was possible to derive the capacity horizontal shear that allowed the identification of the peak ground acceleration related these limits (PGA). The demand with which to compare is, instead, obtained from the elastic response spectra in terms of acceleration as a function of the structure's own period. The results regarding the initial state are presented in Table 3.

It can be noted that the building already does not have a good seismic performance, showing greater vulnerability in the transverse direction where resistance is ensured for an earthquake equal to 41% of that expected in that area. The external steel exoskeleton is designed to achieve an increase in the stiffness of the structure and therefore in capacity to collapse.

Limit State	PGA_D – X	PGA_C – X	C/D	PGA_D – Y	PGA_C – Y	C/D
DL	0,076 g	0,103 g	**1,36**	0,076 g	0,073 g	**0,96**
SD	0,184 g	0,111 g	**0,60**	0,184 g	0,075 g	**0,41**
NC	0,244 g	0,111 g	**0,46**	0,244 g	0,076 g	**0,31**

Table 3. PGA values derived from the pushover analysis and related to the considered limit state in both the main directions. The capacity/demand ratio are also presented.

3.4. PROJECT SOLUTION, 3D EXOSKELETON

The project structure consists of a steel frame for each floor, with bracing in the transverse direction, connected to the existing reinforced concrete frame at the beam-column joints. These frames are connected in longitudinal direction with additional beams hinged to create the space suitable for housing the volumetric additions. The connection profile between the two structures is considered rigidly connected (In the model, S275 structural steel was used with the following sections: HEA 300 for pillars, HEA 200 for transversal beams, IPE 160/200 for longitudinal beams, ϕ 76,1x3,2 for vertical concentric braces, ϕ30x2,9 for horizontal concentric braces and ϕ193,7x4,5 for the connecting pipes between the two structures.).

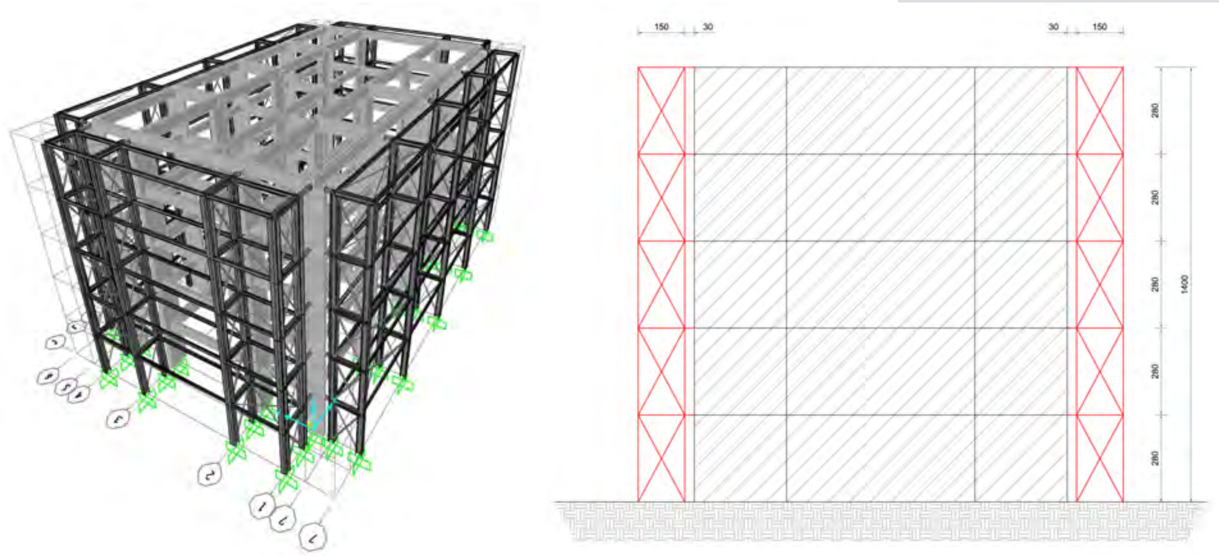

Figure 3. Finite element model (on the left) and geometrical section scheme of the structural addition (on the right).

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This connection consists of a steel profile (stiff in both the reference axes) connected to the exoskeleton by means of a flange and connected to the concrete joint with an UPN profiles fixed along the perimetral beams.
It represents the situation closest to the rigid joint simulated, even if it is difficult to realize.
The application of the structure on the entire perimeter of the existing floor has been studied in order to guarantee a reinforcement in both directions, as shown in Fig. 3.
The modal analysis following the application of the exoskeleton shows that the main vibrating mode in Y (U2) transverse direction, maintains an activated mass percentage around 75,7% with a period reduced to 0,488 s. The maximum displacements in the DL limit state correspond to 1,5 cm in longitudinal directions (X) and 1,6 cm in the transversal ones (Y) showing a reduction of respectively 23% and about 40%. Finally, using the same settings used previously, the capacity curves of the structure were recalculated, and it was possible to derive the new shear values for the determination of the PGA. In Table 4, the comparison is shown after the application of the external steel structure.

Limit State	PGA_D – X	PGA_C – X		C/D	PGA_D – Y	PGA_C – Y		C/D
		Before	After			Before	After	
DL	0,076 g	0,103 g	0,170 g	**2,24**	0,076 g	0,073 g	0,158 g	**2,08**
SD	0,184 g	0,111 g	0,200 g	**1,09**	0,184 g	0,075 g	0,200 g	**1,09**
NC	0,244 g	0,111 g	0,200 g	**0,82**	0,244 g	0,076 g	0,200 g	**0,82**

Table 4. PGA values derived from the pushover analysis and related to the considered limit state in both the main directions. The capacity values before and after the exoskeleton application have been compared. The new capacity/demand ratio are also presented.

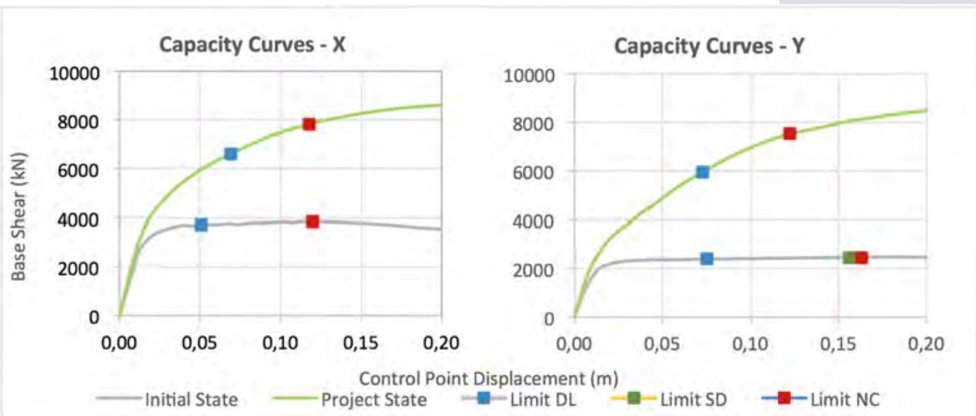

Figure 4. Capacity curves of the structure in the two directions before and after the intervention. The points where the limit states are exceeded are indicated.

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A substantial increase in the collapse acceleration of the structure can be seen with respect to the demand value which remains constant as the periods of the vibrating modes always fall in the portion of the spectrum with constant acceleration. The strengthening guaranteed by the steel structure is significant and allows the achievement of the acceleration demand value. The capacity curves obtained are shown in Fig. 4.

The improvement of the capacity of the structure must be followed by a rigorous verification of capacity/demand in terms of displacement, performed in this case with the method of the target displacement described both in the Eurocode 8 [8] and in the circular of the Italian technical standards [9]. As already mentioned above, the increase in stiffness leads to a reduction in the overall ductility of the structure which requires an accurate evaluation of the displacements. Table 5 below, shows the displacement values of demand and capacity for the X direction.

Limit State	δ_D (mm)	δ_C (mm)	D/C	δ_D (mm)	δ_C (mm)	D/C
	Initial state			State of project		
DL	12,36	50,69	**24%**	21,39	69,00	**31%**
SD	29,52	120,23	**25%**	51,08	117,70	**43%**
NC	47,26	120,23	**39%**	67,95	117,70	**58%**

Table 5. Capacity and demand displacement values calculated on the single degree of freedom equivalent system (SDOF) along the X direction. The D/C ratios of the initial and project state are compared.

It can be seen how the application of the external steel structure goes to increase the D/C values which consequently must be checked in parallel to the capacity increases in terms of acceleration.

4. TWO-DIMENSIONAL EXOSKELETON, GRONINGEN CASE STUDY

The Ducth case studies are buildings part of blocks of typical Martini K terraced houses which were built in the 1960s when no seismic requirements were considered in the Groningen area. Therefore, the houses are designed to mainly sustain vertical loads with only marginal horizontal loading from wind. The applied strengthening method consists in the application of a steel portal in the longitudinal direction with wood skeleton walls and insulation. The calculation of the steel portal frames assumes that the entire stabilizing function in X direction is taken over by the new steel exoskeleton. The influence of masonry walls is neglected. However, it will have to be demonstrated that the load-bearing walls can continue to fulfil their function in the deformations occurring in the NC-limit state.

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4.1. INDUCED SEISMICITY

The on-going process of accurately assessing the ground conditions and the peculiarities of the Groningen area that influence the design and retrofitting of buildings is very dynamic and assumes very frequent modifications and new releases of the national guidelines, NPR 9998 [10]. The peculiarities of the seismic area of Groningen are related mainly to the nature of the seismic events occurring here. Unlike the regular tectonic earthquakes, the ones in Groningen are of induced nature, caused by the gas extractions from the ground in the area. This fact influences both the depth at which the hypocenter is located and hence at which energy is being released as well as the spectrum of accelerations that the ground surface experiences.

One of the most important features is the nature of the seismic event as it directly influences the depth of the hypocenter. It is commonon knowledge that generally tectonic earthquakes have considerably deeper hypocenters than induced earthquakes (see Fig. 5). This fact influences the area on which the waves are spread at the surface of the ground. In fact, the energy is released over a much smaller area with serious consequences as higher values of peak ground accelerations are generated.

The intensity of an earthquake is measured in the well-known Richter scale. The magnitude relates to the energy that is released by the earthquake. Because of the shallow depth of the hypocentre it can be observed that for the same magnitude the accelerations and therefore the damage of an induced earthquake will be higher than that caused by a deeper tectonic earthquake.

Another main factor to be considered is the duration of the event in relation to the acceleration values.

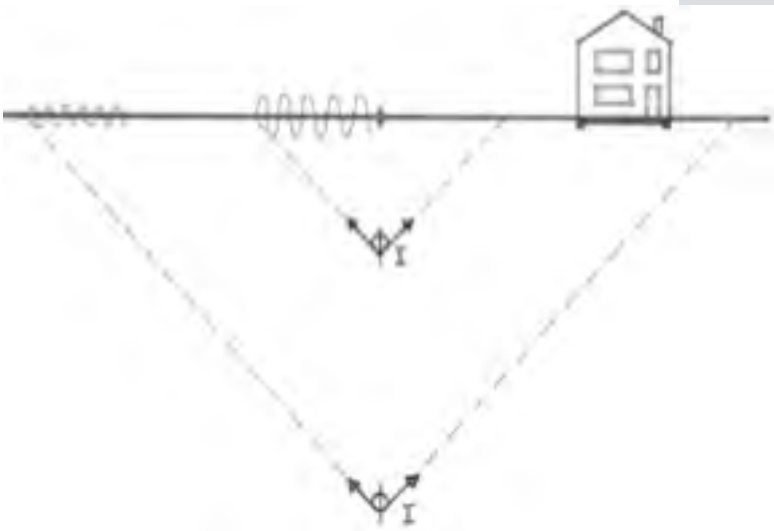

Figure 5. Influence of the hypocentre depth on the surface area and on the value of peak ground accelerations. The deeper the hypocentre, the greater the area of diffusion of the seismic energy and the lower the value of the acceleration.

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In case of an induced earthquake, for example Huizinge earthquake in 2012 (B.Dost, D.Kraaijpoel, [11]), the shaking motion is of shorter duration (typically 1-2 seconds) and with higher frequency than in case of natural Dutch earthquakes, such as the earthquake in Roermond from 1992 (T. Camelbeeck et al. [12]) , with a recorded time signal of 5-10 seconds. These result in less damage for this type of induced earthquake in Groningen, compared to a tectonic earthquake with the same peak ground acceleration. After considering all these aspects, the procedure for determining the seismic action ends with the determination of the response spectra, specific for the Groningen earthquakes.

4.2. ANALYSED BUILDINGS AND STRUCTURAL MODELLING

The houses are typical unreinforced masonry structures with pitched gable roof. The ground level floor is made of wooden slab with planking, the first level and the attic consist of a cast-in-place concrete slab with a thickness of 110 mm. The bearing walls, front and back facades, are mostly build of masonry bricks, but can also be made of calcium silicate or concrete bricks. No basement is present, and the foundation system can sometimes be a shallow foundation on sand, but also piled foundations are very common where the subsoil is weaker. The Structure is mainly designed to take vertical loads.
The masonry of the bearing walls is sometimes made of calcium silicate, with modules E = 3500 Mpa and G = 1450 Mpa, a compressive strength of 7 Mpa and the maximum shear strength of 0,78 Mpa (with an initial ones of 0,25 Mpa). But the bearing walls can also be made of regular masonry or concrete bricks. A layer of red brick is applied for the outer leaf of the facades.
The triangular top walls consist of an inner cavity sheet of 100 mm stone. The house-dividing top walls are made up of two separate 100 mm thick walls with

Figure 6. On the left, the plan of the terraced houses with identification of the walls thickness. On the right a photo of the houses.

a 30 mm cavity. The concrete floors, made with K225 (C13/16 with a reference value of 13 MPa) and QR24 or QR40 for reinforcing steel (respectivily 240 Mpa and 460 Mpa), are not continuous for the row of houses, but separated by individual living unit and connected in the middle of the 200 mm thick wall. Below two pictures of the terraced houses (Fig. 6).

Concerning existing construction, which must be statically tested based on the definitions in NEN-8700, this static evaluation is not in the scope of the project. The seismic test criteria follow from NPR 9998 [10] and NEN-EN-1990 [13, 14 and 15]. For the determination of dynamic behaviour, the building is modelled in two main directions into a mass spring system, each mass representing a building floor level. The masses are connected by elements with bending stiffness, which represent the stability system between the floors. The mass spring system is supported by a rotational spring, whose stiffness is determined according to the foundation conditions, as can be seen in Fig. 7.

Figure 7. Principle of the mass spring system.

4.3. INITIAL STATE

The initial structure is analysed by means of a spectral modal calculation. The most important output of the spectral modal analysis is the highest horizontal force. The building-specific assumptions required for determining the boundary conditions for the calculation of existing structures are named in table 6. These considerations are according to NPR 9998 [10].

Item	DL	SD	NC
▪ Importance class		CC1b	
▪ Peak ground acceleration $a_{g,ref}$ in [g] at ground surface with a return period of 475 years	0,30 g	0,30 g	0,30 g
▪ Importance factor γ_i for existing buildings, according to NPR February 2015, individual risk level 10^{-4}	0,52	1,0	1,2
▪ BfD Reduction factor for individual risk level 10^{-4}	67%	67%	67%
▪ “design value” $a_{g,d}$ for the peak ground acceleration	0,10 g	0,20 g	0,24 g

Table 6. Reference data for determing the design value of acceleration to derive the seismic action for the initial state.

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It needs to be noted that this is the NPR 9998 of 2015. Knowledge of the seismic risk has advanced considerably since then.

This means that it is now known that the actual seismic risk is much lower than what is shown here. Furthermore, the Dutch government has taken action in reducing the gasproduction, which has led to a foreseen considerable further reduction of the seismic risk.

The difference between the horizontal load in each direction is caused by the difference in stiffness per direction. The horizontal seismic loads, shown in table 7 (included in the point masses of the floors), must be transferred to the foundation through the stability system.

Horizontal direction	Load amplification factor due to second order effects	Horizontal seismic force [kN]
x-richting	1,00	970
y-richting	1,00	2820

Table 7. Horizontal seismic force applied in the initial state condition.

The calculations show that the seismic load is relatively low in x direction and high in y direction. The shear capacity of the masonry is exceeded, and the stability elements do not verify the requirement in the x-direction (see table 8).

Horizontal direction	Element	Needed capacity [N/mm2]	Calculated capacity [N/mm2]	Unity check
X	masonry	0,46	0,15	3,1 – does not comply
Y	masonry	0,26	0,26	1,0 – complies

Table 8. Verification of the shear capacity of the masonry piers.

This low stiffness causes the largest percentage of the mass of the building to be brought into a relatively low-frequency vibration, with relatively low horizontal seismic loads occurring. This vibration is associated with relatively large horizontal displacements.

The bearing walls out of plane loads are also checked for calculating the bending moment due to the earthquake load. A behaviour factor is calculated according to Table 9.2 of the NPR [10]. The masonry is considered according to NEN-EN 1998-1 [13]. Wall calculations investigate the requirements of slenderness and strength. The 100 mm end walls are not verified, these walls need to be strengthened. The gable walls are not considered in the calculation, however, in the current situation these are only supported by the rafters of the roof, because they are not properly anchored to the wall. As a result, the top wall will come loose.

An alternative is the use of Non-Linear Pushover Analyses to more accurately describe the specific response of lowrise masonry structures. The very pronounced non-linearities are otherwise not captured adequately in the modal response spectrum (MRS) calculation.

4.4. PROJECT SOLUTION, 2D EXOSKELETON

The aim of the intervention is to seismically strengthen a large number of row-houses and surroundings against the influence of earthquakes. For this purpose, the increase in safety is governed by the will to minimize the influence on inhabitants, the duration of the approach keeping the costs low and improving the sustainability of the houses. The constructive behaviour of the building after these reinforcements complies with the seismic requirements of NPR 9998 [10], NEN-EN 1998 [13, 14 and 15]. The reinforcement includes the following aspects:

- Steel frame in X direction, the planar exoskeleton (An S235JR steel was used with $f_{yk} \geq 235$ MPa and $f_{tk} \geq 360$ MPa);
- Strengthening of out of plane end façade masonry;
- Required strengthening of gable walls out of plane.

The steel portal in X direction is designed such that it can plastically deform to withstand the spectral deformation induced by the earthquake. The steelframes are calculated with a non-linear pushover analysis. The structural model consists of columns to which steel beams are rigidly connected. For each house two portals at the front and two portals at the back are applied. The portals are stacked and hinged with each other. Each of these takes into

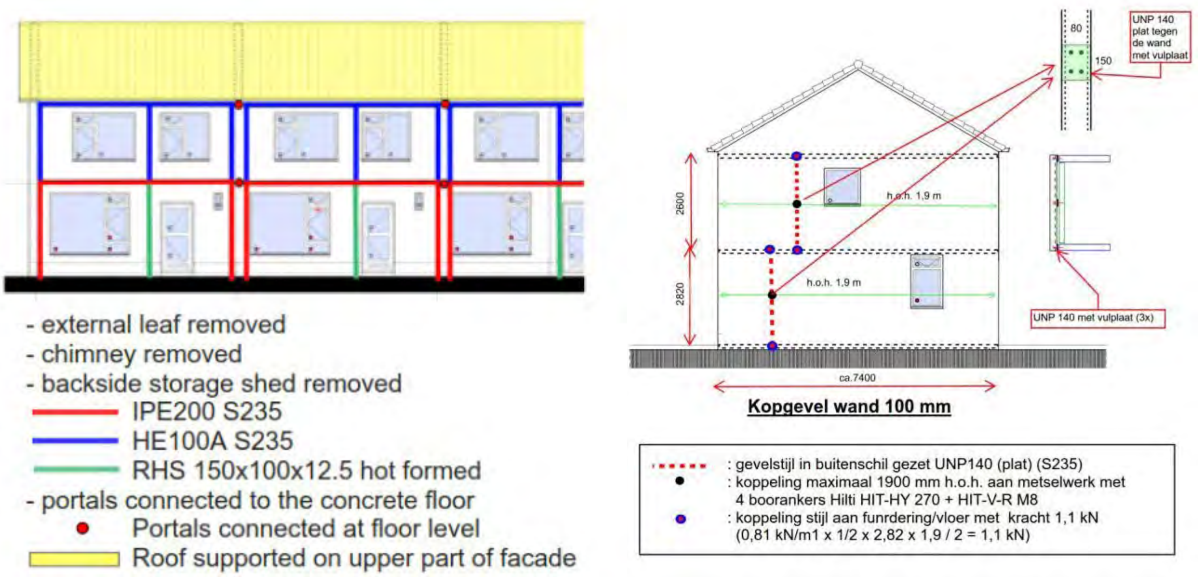

Figure 8. Façade schemes of the project strategies, on the left the planar exoskeleton geometrical scheme and on the right the reinforcement against the out of plane risk of the end walls.

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account two connection principles for transferring the horizontal force in X direction. The U-section that connects the steel frame to the floor is rigidly connected to the steel frame but has a hinged connection to the concrete floor. This way any bending moment caused by the eccentric connection will only give bending moments in the steel frames and not in the (fragile) anchore-connection to the concrete floor. It is the most important and difficult part of the strengthening as the connection needs to transfer large forces to a very thin concrete floor. Furthermore, the connection needs to cope with large tolerances, as the exact measurements of the façade only become apparent after demolition of the outer wythe of the cavity wall. By that time, the new portalframes and their connections are already on site and need to be able to be adjusted to fit.

From calculation of the initial structure it is concluded that the end facades with the 100 mm thick masonry do not comply out of plane. These walls are supported out of plane by vertical struts connected to the main steel frames. Also, the gable walls are insufficiently supported in the longitudinal direction (Y). Thus, to prevent out of plane failure due to a seismic load, a horizontal support is applied by ensuring that the horizontal loads of the roof are transferred to the attic floor.

The type of analysis varies depending on the reinforcing measure. For the portals in X direction a push-over analysis without FEM software is performed. Taking into account the overcapacity factor and the knowledge factor, maximum forces on the connections are determined (factor 1.375x1.2 = 1.65).

The existing structure takes care of static loads like permanent loads (structural element own weight, permanent structural and no-structural weight) and variable loads (live, snow and wind). The new steel structure deals with horizontal actions due to the earthquake. The building-specific assumptions required for determining the relevant parameters for the calculation of strengthening measures are listed below in table 9.

The assumptions are according to NPR [10].

Item	DL	SD	NC
▪ Importance class		CC1b	
▪ Peak ground acceleration $a_{g,ref}$ in [g] at ground surface with a return period of 475 years	0,42 g	0,42 g	0,42 g
▪ Importance factor γ_i for existing buildings, according to NPR February 2015, individual risk level 10^{-4}	0,50	1,0	1,2
▪ BfD Reduction factor for individual risk level 10^{-4}	67%	67%	67%
▪ “design value” $a_{g,d}$ for the peak ground acceleration	0,14 g	0,28 g	0,34 g

Table 9. Reference data for determing the design value of acceleration to derive the seismic action for the state of project.

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The seismic analysis of the existing structure is used as a starting point for the seismic strengthening. The main improvement lies in the fact that the structure is now capable of taking horizontal seismic forces in X direction by means of the new steel exoskeleton. At the same time, the prevention of the out-of-plane movement of the end walls was achieved by ensuring the roof system is able to transmit the horizontal loads towards the foundations. Concluding, the proposed specific interventions lead to the achievement of the threshold imposed by the verifications and therefore to resist the seismic action considered.
The main interventions have been used to also improve the energetic quality of the buildings. The steel frames were pre-manufactured with a complete insulation skin. The same was done with the roof panels, which also were fitted with solar panels. This way the houses were converted to zero-energy buildings.

5. CONCLUSIONS

The work presents the first results obtained within the ongoing research activity aimed at studying an intervention technique able to solve in an integrated way the well-known structural and energetic shortage of the existing buildings constructed in the II post-war period. Following the definition of the seismic action, underlining the possible differences arising from the characterization of the territories involved in the European project Pro-GET-onE, two cases of structural seismic improvement have been presented through the application of exoskeletons. The possibility of reinforcing existing structures has been verified through the application of steel frames directly connected to existing structures. These additions can be evaluated in the existing wall plane or perpendicular to it, in relation to the typhology and behavior of the existing

Figure 9. Pictures of the 2D exoskeleton applied on the masonry terraced houses.

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structure. In the two cases presented, there is the possibility of significantly improving seismic performance. In the first, going to increase the capacity in terms of collapse acceleration, evaluated on the overall behavior of the reinforced concrete building. While in the second through interventions and analyzes aimed at satisfying the verification of all the members.
The interventions were not aimed to improve the energy performance of the buildings, but with these big intrusions to the current buildings these measures could be added to the programme with relatively limited costs (but big energetic gains).

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