Seismic performance evaluation of an irregular RC frame building

L'expertise de la performance séismiques d'un immeuble éxistant en BA avec la structure de géometrie irrégulière

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ABSTRACT This paper focuses on overcoming difficulties associated with the assessment of a multi-story reinforced concrete frame building constructed in several phases over time, with an irregular layout. Trying to develop a coherent, safe and sensible design in such a case is indeed a challenge for structural and geotechnical engineers. The retrofitting problem is made worse by incomplete data, a building with shallow and deep foundations, and architectural "re-purposing" of the entire structural assembly. A state-of-the art seismic performance evaluation according to Eurocode 8-3 is presented as a case study, with special emphasis on the topics of numerical modeling and foundation modeling. Potential modeling simplifications are considered as well.

RÉSUMÉ Dans cette étude on va présenter les aspects spécifiques à affronter par les ingénieurs géotechniciens lors de leur activité, à voir la réponse de la structure aux effets séismiques. Dans les cas en question, c'est l'obligation primordiale de présenter des plans cohérents de toute sécurité, et sensibles à tous les effets possibles. L'analyse de la structure a été particulièrement laborieux à cause des défaillances des documents existants, par les systèmes de fondation mixte, ainsi que les modification totale des fonctions architecturales. En profitant de l'occasion, on va également présenter l'analyse des réactions séismiques calculées suivant les normes Eurocode 8-3, tout en soulignant les difficultés de modellisation numérique et des fondations. On a également analysé les possibilités de simplification de la procédure de modellisation.

1 INTRODUCTION

In Hungary, previous building codes regulated the seismic design of engineering structures less strictly than the current Eurocode provisions. As a result of the transition to Eurocode, structural and geotechnical engineers now have to face new challenges i.e. seismic design of new buildings and the assessment and potential retrofitting of existing buildings. The latter topic may even be more difficult, since most of our buildings have been designed without considering conceptual topics connected to seismic performance. Hungary's seismicity can be considered moderate compared to the neighboring countries, however based on historical data and the seismologist's researches (Tóth et. al. 2006) earthquakes do indeed occur in Hungary with enough regularity and intensity to warrant careful attention to the possibility of seismic actions. According to the map of seismic zones in the Annex of Eurocode 8-1 the PGA varies between 0.08g - 0.15g for 10% probability of exceedence over 50 years.

2 EUROCODE 8-3

Part 3 of Eurocode 8 deals with assessment and retrofitting of buildings for earthquake loading. As discussed in many papers and books (e.g. Fardis 2009) Eurocode 8-3 is a unique part of the Eurocode standard package in many ways. It is the only part dealing with existing structures, but unlike all other Eurocodes it only applies to those structures, that the owner or local Authorities have decided to seismically assess and possibly retrofit. Hence the extent and scope of the normative, mandatory part of Eurocode 8-3 is quite limited and also the Annexes give only informative details and material specific aspects for the different type of structures. Since the assessment and retrofitting of existing buildings may require relatively large costs, the necessity of any action on the existing building stock should be decided on a national level, based on economic studies. Naturally for a country with no extensive historical knowledge and design practice connected to seismic design, all information given in the code is a really good basis. However, without the mentioned studies, the use of these design and assessment methods may be very uneconomic on the global scale, yet e.g. in Hungary there are no alternative national sources of information available.

The necessity of the assessment at the moment in Hungary is regulated relatively simply, but quite strictly. According to the regulations (Dulácska et al. 2013), the seismic assessment of an existing, max. 4 story high building can be neglected, if “no existing (10 cm thick or thicker) walls have been removed or have been significantly weakened; or if their missing stiffness is substituted by another structure’s stiffness”. In other words, whenever the renewal of such a building is planned, if any walls have been weakened or removed, the assessment and retrofitting should be carried out. The latter part of the condition is especially interesting, because if the missing stiffness has been substituted, the assessment can be disregarded, although a stiffer structure may even suffer more damage in a potential earthquake, than the building in its original state. In the Author’s opinion the assessment in such a case may even be more important.

Eurocode 8-3 introduces three Limit States (Near Collapse, Significant Damage and Damage Limitation) and corresponding hazard levels and it is left to national decision, which of these should be met. If the task is only to assess an existing building’s seismic performance, the design ground acceleration may be reduced based on the remaining lifetime of the building. If the task is to renew and retrofit the building, this reduction cannot be used, since in this case the designer is considering a new planned lifetime of e.g. 50 years and not a remaining period of the originally planned lifetime.

The assessment method based on EC8-3 is discussed in more detail in connection with the case study presented here.

3 CASE STUDY

3.1 General data

The object of this case study is a building renewed within an ongoing larger project in Hungary. The building is a multi-story train station built in 1975. Its total floor area is ~ 13 000 m^2 throughout the basement, ground floor and the two stories. The main structure of the building is a reinforced concrete frame, with masonry infill. The many functions of the building had a direct effect on the conceptual design, e.g. the main entrance leads into a large entrance hall with enormous space, where travelers can buy tickets or wait for the train. This entrance hall extends from the ground floor to the roof of the building and the facade is built with multi-story high windows. Other parts of the building contain smaller rooms for railway workers and some larger rooms for traffic control, storage etc. The structural model of the building can be seen in Figure 1.

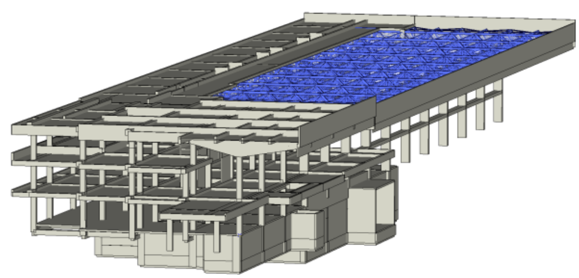

Figure 1. Structural model of building.

Under the ground floor there is a partial basement that extends to one third of its base area. The basement is built on a very stiff reinforced concrete slab while all other parts of the building are supported on pile foundations. The foundations are connected horizontally with reinforced concrete grade beams.

The original plans made available were not comprehensive so a thorough survey had to be completed in order to start planning the renovation design. However, many structural details remained unknown.

From the geological literature Holocene fluvial layers are typical for the site to great depth, soft layers of low plasticity cohesive soils and granular soils alternate each other. The groundwater table is usually close to the surface level. Geotechnical investigations were carried out in order to obtain information about the physical properties of the underlying soil. The investigations consisted of borings and seismic cone penetration tests (SCPT). The latter was performed with a piezocone penetrometer probe equipped with accelerometers to detect shear and pressure waves produced by an impact source at the ground surface. The shear wave velocity measurements obtained from the SCPT tests are shown in Figure 2. They are considered typical for soil conditions in this part of Hungary.

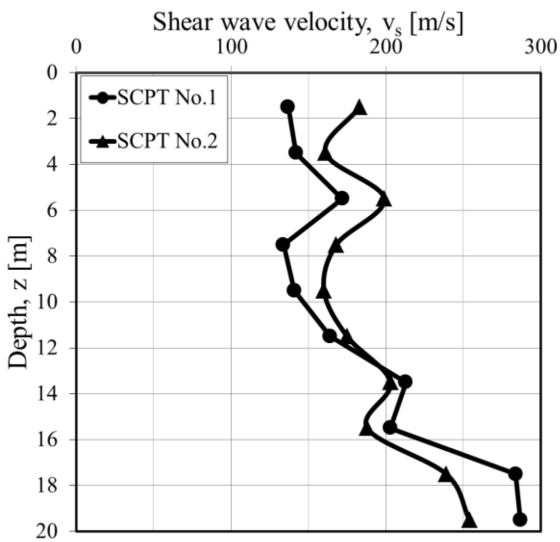

Figure 2. Shear wave velocity profiles of the site.

3.2 *Assessment method*

The assessment began with determining the performance requirements and compliance criteria. As mentioned before the limit states to be considered and their corresponding hazard level is given in the National Annex. In Hungary only the Serious Damage (SD) limit state shall be fulfilled with the suggested hazard level of 10% exceedance in 50 years. The design ground acceleration can be determined based on the importance class and possibly reduced if the building has a remaining lifetime shorter than 50 years. Since the main part of the building is an assembly hall, importance class III was chosen. No reduction was applied to the design ground acceleration because the aim of the project was to retrofit the building with a new planned lifetime of 50 years.

After this, the knowledge level (KL1-KL3 in Eurocode) of the building shall be assessed based on available information about geometry, material and details of the structural system. This means all available data on the properties of the structural elements, the reinforcement in RC members and connections between steel members as well as the mechanical properties of the constituent materials will be gathered and assessed. The knowledge level is based on the completeness of data as limited (KL1), normal (KL2) or full knowledge (KL3). Confidence factors are then assigned to each level; these factors are applied to material properties (strength) in the analysis, a higher knowledge level translates to more confidence in the behavior of the structure and a smaller reduction. Hence this step should include a cost assessment as well; the costs of in-situ inspections should be compared to the possible cost saving of retrofitting based on a higher knowledge level. The knowledge level also influences the type of analysis: sophisticated, non-linear analyses (pushover analysis, time history analysis) should not to be used if the knowledge level is only KL1, i.e. limited knowledge.

Based on the extent of in-situ tests and the incomplete original construction drawings, the project was categorized into the KL2, normal knowledge level.

3.3 *General modeling aspects*

The type of the analysis is usually chosen based on structural properties and available data. In most cases the designer starts with the most simple analysis method (lateral force method) and uses more complex methods only if the results are unsatisfactory. However, the lateral force method may only be used, if the criteria for its applicability given in Eurocode 8-1 are met. These criteria require regularity in plan and elevation, structural conformity etc. This building with its irregular layout, structure and foundation system clearly violates these rules, therefore the next level analysis, a 3D modal response analysis was chosen.

As a first step, the main parts of the building were identified, which could be considered as structurally independent. This is necessary to reduce the computational effort needed to run the dynamic analysis. The different parts will behave independently under the seismic loading, and therefore their separation is essential. However, the possible collision of the separate building parts should later be evaluated.

Knowing that the different areas and rooms of the building will have new functions, the self-weight of the building, combined with the variable actions according to Eurocode 8-1, had to be analyzed based on the new architectural layouts and not on the original plans. This step usually requires an early cooperation between the structural designers and architects, since these functions are not yet decided at the time of the structural analyses. It is advantageous if the refurbishments result in a smaller total mass of the building, since the amplitude of the earthquake load is directly proportional to mass, and also the static design requirements are reduced if this is the case. However a significant reduction is usually difficult to achieve, even with new and advanced technologies and materials. In this case, the total mass remained in the same range as prior to the refurbishments.

Table 1. Determination of average shear wave velocity v_{s30} based on SCPT No. 1.

Layer No.	Top level	Bottom level	Thick-ness	Average velocity	Travel time
	[m]	[m]	h [m]	v_s [m/s]	$t = h/v_s$ [s]
1	0,0	4,6	4,6	140	0,03286
2	4,6	6,8	2,2	172	0,01279
3	6,8	10,5	3,7	138	0,02681
4	10,5	12,2	1,7	164	0,01037
5	12,2	16,2	4,0	207	0,01932
6	16,2	20,0	3,8	285	0,01333
7	20,0	30,0	10,0	285	0,03509
	Total thickness		30,0 m	Total time	0,15057 s
		v_{s30} = Total thickness / Total time =			199 m/s

In addition to the structural properties of the building, the earthquake load in the modal response analysis is dependent on ground conditions. The top 30 meters of the subsoil should be evaluated. Based on the SCPT measurements, two shear wave velocity profiles were obtained and v_{s30} values were determined as shown in Tables 1 and 2. While the depth of the SCPT's was only 20 meters, the total 30 m depth could be assessed. Extrapolation was based on previous soil investigations in the area of the site as well as geological literature. From the additional data, a constant shear wave velocity was assumed equal to the SCPT value for the final 10 meters (Layer 7).

Table 2. Determination of average shear wave velocity v_{s30} based on SCPT No. 2.

Layer No.	Top level	Bottom level	Thick-ness	Average velocity	Travel time
	[m]	[m]	h [m]	v_s [m/s]	$t = h/v_s$ [s]
1	0,0	4,0	4,0	172	0,02326
2	4,0	6,3	2,3	199	0,01156
3	6,3	11,8	5,5	168	0,03274
4	11,8	14,8	3,0	203	0,01478
5	14,8	17,2	2,4	188	0,01277
6	17,2	20,0	2,8	247	0,01134
7	20,0	30,0	10,0	247	0,04049
	Total thickness		30,0 m	Total time	0,14692 s
		v_{s30} = Total thickness / Total time =			204 m/s

Without this addition, the average shear wave velocities for the top 20 meters were 8-15% lower. Based on the investigations, the soil was classified into ground type C according to Eurocode 8-1.

Another important modelling aspect is the stiffness of reinforced concrete elements and their effect on global structural response. Eurocode 8 provisions for energy dissipation and ductility were developed based on the global inelastic behavior of structures to monotonic lateral (earthquake) loading, which is usually presumed bilinear, close to elastic-perfectly plastic. Therefore the effect of cracked sections must be taken into account and the assumed RC members (usually vertical structural elements) should be modelled with a reduced effective stiffness. As an initial value, a stiffness of 50% of the uncracked member can be used according to the code. This estimated cracked stiffness is still quite high; experiments show even lower values. During the performance evaluation of the members, the effective stiffness can be calculated and the analysis can be refined. Lower stiffness of vertical members will result in lower member forces but at the same time produce larger deformations. Hence lateral drifts and P-Δ effects must be carefully inspected.

3.4 Foundation modelling

The modelling of the foundation elements (pile groups and spread foundation of the basement) was particularly difficult and had a significant effect on the structural performance.

Traditionally, modelers did not include foundation elements with the superstructure. They considered the superstructure fully fixed at the top of the foundation system and the foundation was designed separately to carry the calculated support forces and moments. The disadvantages of this approach have been discussed by many authors and the coupled modelling of the foundation and the superstructure can have a very beneficial effect (Ray and Wolf, 2013). However, in many cases this method is still not feasible, especially in everyday practice. If it is not feasible one may consider the soil compliance carefully and use elastic support elements (Winkler springs) with realistic stiffness and a thorough sensitivity analysis. This approach was taken for the study also.

The raft foundation was supported by elements defined with subgrade reaction modulus, based on the work of Gazetas, given in (ASCE 2007). The basement itself consisted of a very stiff base plate, RC columns and many walls. While the architectural functions on the ground level required large spaces, the basement was mainly used for storage and was designed to be much stiffer. In the early stages of the dynamic analysis, the basement was included in the structural model with all its elements. However, even after many attempts and with different configurations it was not possible to reach the required 90% modal mass for the modal response analysis with the basement level in the model. The stiff basement box did not suffer the magnitude of deformations compared to higher levels of the building, while its mass was considerable.

Pile foundation design for seismic loading has also been a challenging research topic for over 50 years. The modelling of single piles under lateral loads with different approaches has evolved greatly. Early recommendations for pile response suggested a subgrade reaction approach with linear or non-linear springs to mimic the lateral support of soil, and dashpots for modelling both material and radiation damping. Recently, more complex methods, such as finite element approaches, are more widely used. Computational cost (usually considered as employee time and lost utility of computer resources) is still a barrier for these calculations. Modelling pile groups is even more difficult and has not yet been integrated into the structural design work-flow. Eurocode 8-5 gives a method for calculating pile head stiffness for a single pile for different soil stiffness profiles. Pile groups can either be studied in a separate model, considering typical load combinations and a resultant stiffness applied to the structural model; or pile group stiffness can be calculated based on geometry and single pile stiffness by using a group factor. In this study the structural FEM model consisted of ~28 000 plate elements and ~10 000 beam elements. Therefore, it was not feasible to model pile groups with more beam elements; a pile group was modeled with a single fixity.

The pile group stiffness for horizontal and vertical movement can be taken as the sum of those of the individual piles. The group rotational stiffness e.g. about the horizontal x axis can be calculated as:

$$K_{xx} = \sum_i (y_i{}^2 k_{zi} + k_{xxi}) \tag{1}$$

where

y_i is the distance of pile i form the x axis,

k_{zi} is the single pile axial stiffness, E_cA/L,

k_{xxi} is the single pile rotational stiffness.

Pile foundation stiffness has a significant effect on the structural behavior (Ray and Wolf, 2013). Table 3 shows the effect of pile group stiffness on the period of the first three modes.

Table 3. Effect of pile foundation stiffness on periods

Pile stiffness range	Mode No.	Frequency	Period	Modal mass contribution	
[kN/m], [kN/rad]	-	[Hz]	[s]	ε_x	ε_y
~1*10^5	1	**1,76**	**0,57**	0,012	0,206
	2	1,94	0,52	0,004	0,624
	3	2,33	0,43	0,830	0,000
~5*10^5	1	**1,98**	**0,51**	0,005	0,255
	2	2,49	0,40	0,047	0,543
	3	2,64	0,38	0,769	0,020
~1*10^6	1	**2,07**	**0,48**	0,003	0,239
	2	2,73	0,37	0,175	0,458
	3	2,82	0,35	0,621	0,095

Note: only the first 3 modes are listed here, in the calculation 40 modes were considered.

The fundamental mode of this building was torsional; the second and third modes were translational modes in y and x direction respectively. All periods decreased with increasing pile foundation stiffness, while the earthquake loading and shear forces increased in the vertical structural elements.

3.5 *Performance evaluation*

The Annexes of Eurocode 8-3 provide information about calculating resistances of reinforced concrete, steel, and composite structural members, as well as masonry buildings. Criteria are defined for each limit state. Strengthening methods and their performance are also discussed. These calculation procedures were developed based on observed behavior of buildings during earthquakes and many laboratory tests and extensive research. Understanding and using these methods is challenging, especially for designers who are traditionally not accustomed to seismic performance evaluation. For a thorough discussion about the calculation methods and their background, the reader is referred to (Fardis 2009).

In this study, the most critical elements were the columns. These elements are often critical for earthquake loading, if at the time of design no such loading was considered. Their behavior is quite complex. Two main failure modes have to be assessed: the deformation capability shall be verified (defined with the chord rotation); and the shear resistance should be evaluated. If shear failure precedes flexural yielding, a brittle failure mode is dominant, but in some cases concrete members that first yielded in flexure failed after the cyclic degradation of their shear strength.

As a consequence of the irregular layout, the fundamental mode of the building was torsional. Shear forces and moments were especially high in the columns at the shorter sides of the building. The original layout didn't consider lateral loads at this level; hence no significant lateral bracing was designed or built. Due to the multiple functions of the building, a simple and regular bracing system was not possible; many different retrofitting arrangements had to be assessed.

As a result of the performance evaluation, the strengthening of the columns with steel jacketing was chosen. Additionally, thanks to the planned functional refurbishments, new shear walls could be added to the layout contributing to the global resistance of the building.

4 CONCLUSIONS

Seismic assessment of existing buildings and their possible retrofitting is a new and challenging topic for Hungarian structural and geotechnical engineers. Clients, project leaders and design companies up to now haven't been aware of the time and effort needed to perform such seismic performance evaluations. In addition, the largest part of our current building stock has been designed without considering seismic effects. A case study was presented with emphasis on the problems arising from a difficult and irregular layout. General modeling aspects and difficulties connected to foundation modeling were discussed. Although many researchers suggest that the coupled modeling of foundation and structure is preferable, in many practical cases it is still not feasible. Therefore, it is quite clear that further research is required in this field in order to improve the design process.

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