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OPTIMIZATION OF MODAL RESPONSE SPECTRUM ANALYSIS METHOD FOR EXISTING BUILDING ASSESSMENT OF UNREINFORCED MASONRY BUILDINGS IN GRONINGEN

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Abstract: *In the Groningen region of the Netherlands, earthquakes have been triggered from gas extraction activities. To guarantee safety, existing buildings in affected areas are consequently assessed by the recent Dutch guideline, the NPR 9998 (Nederlandse Praktijkrichtlijn). One of the frequently used analysis methods allowed by the NPR 9998 is the Modal Response Spectrum Analysis, MRSA; where the use of the Linear Static Analysis method is usually not suitable for most existing structures due to its irregularity limitations. Several structures have been evaluated within the VIIA-Groningen project by means of MRSA. Here, difficulties arose from rationally applied conservative assumptions and demand/resistance factors to the analysis. These difficulties can be explained by the uncertainties related to the behavior of structures located in initially non-seismic regions facing a new magnitude of the hazard. These resulted in more retrofit measures than necessary, bringing up the necessity in occasions to conduct higher complexity analysis, for instance non-linear time history analyses, NLTHA. The effectiveness of retrofit cost reduction from NLTHA is evident after comparing database cost data for buildings that had been evaluated by both MRSA and NLTHA methods for unreinforced masonry structures, URM, a common typology in the Groningen area. This article, presents various optimization observations for MRSA for URM structures, based on the experience of the application of the NPR 9998 and the comparison of buildings with MRSA and NLTHA assessments. Results make evident the impact of realistic modelling in aspects such as soil-structure interface, structural components and initial assumptions in the MRSA assessment.*

Introduction

Since the year 1986, seismic activity due gas extraction activities has been observed in the Groningen province in the North of the Netherlands, with not major public attention up to the Huizinge earthquake, in 2012, when earthquakes became recognized as a potential safety risk (NAM (2015)). Prior to the gas extraction activities, the region of Groningen has been considered as a very low seismicity area; in fact, located in the zone with the lowest expected seismic intensity for the country (about 0,01g according to Solomos et al., 2008).The seismic hazard intensity to be used for the design and evaluation of structures has been a topic of debate since, different from usual tectonic earthquakes, it is dependent on gas production scenarios. The most recent production scenarios target to a cessation of production by 2030 (van Elk et al., 2018). Seismic design spectra for three different scenarios, according to the expected retrofitting measures construction execution period, can be found by means of a Webtool (SKWT, 2019); found in a website interface designed to be used together with the recent 2018 seismic provision, the NPR 9998:2018.

Apart from the seismic hazard magnitude discussion, major challenges are to be addressed regarding the expected seismic performance of buildings in the threatened region. Constructions have been designed without following seismic guidelines and many are built up from un-reinforced

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masonry, URM, a well-known high seismic vulnerable construction type (Grünthal, 1998)). In total, 85% of the building stock is considered to be URM (van Elk and Doornhof, 2017)).

Similar to most regulations, the Dutch NPR 9998 guideline mention the linear static, LS, and the modal response spectrum, MRS, as the common methods to analyse seismic actions on structures. Use of the LS method in assessing existing structures is rare since buildings commonly in the region do not comply with the irregularity limitations. This leaves the MRSA as the common assessment method; where non-linear time history, NLTH, and non-linear pushover, NLPO, methods have also been widely used since they provide with economical retrofitting solutions, with drawbacks in terms of greater analysis costs, time and the necessity of higher engineering expertise.

In particular time is an important aspect since the occupied building stock in the hazardous region is of about 150 000 buildings (van Elk and Doornhof, 2017), where detailed assessments are possibly required for most higher importance structures and buildings located in the higher seismic hazardous areas.

Assessment by means of MRSA and NLTHA

Within the context of the VIIA-Groningen project (VIIA project is a joint venture in between Royal HaskoningDHV and Visser & Smit Bouw), several URM buildings had been assessed by means of both MRSA and NLTHA. In occasions, buildings that had been originally evaluated by means of MRSA were also subjected later on to a NLTHA, since the retrofitting measures were considered not feasible. In figure 1, a comparison in between 59 buildings analysed with both methods is shown. There, the total cost of all measures, in Euros to November 2018, are associated to a so called reference base shear, which corresponds to the value of the total mass multiplied by the reference seismic acceleration used for the structural evaluation.

From Figure 1, it is observed that there is a tendency to achieve greater economically advantageous results from the NLTHA as the reference base shear is growing; for instance, increasing from a factor of 2,0 for a reference base shear of 100 tons-%g to a factor of 2,5 for a reference base shear of 1000 tons-%g. It also highlighted well the fact that using methods of higher sophistication leads to cheaper solutions. Nevertheless, it is important to observe that not necessarily the differences came only from advantageous use of non-linear material properties in NLTH, but they also came from conservative assumptions related to the uncertainties in URM structural performance for MRSA. Such conservatism is well justified in principle in the Groningen area, where the engineer is dealing with structures located in a previously non-seismic area.

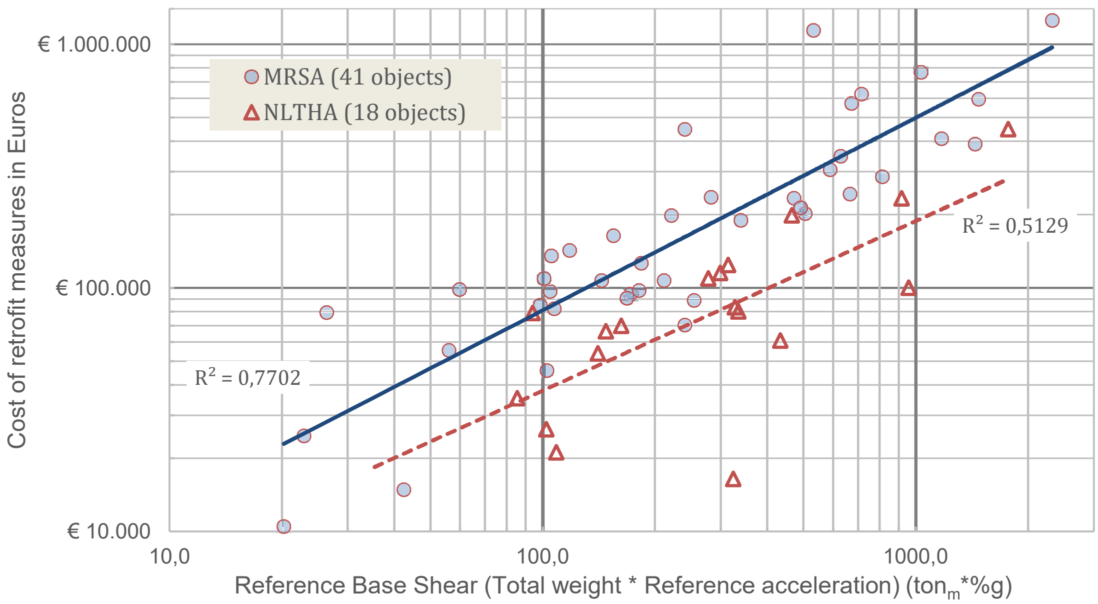


Figure 1. Relation in between seismic retrofitting measures estimated cost and the reference base shear for URM in Groningen (without including cost of the seismic analysis and with a reference peak ground acceleration in between 0,14g and 0,36g for a return period of 475 years)

Results from Figure 1 indicate, in practical terms, that structures can be retrofit in average for at least half the cost of the actual results after conducting MRSA. For some specific cases, like clay URM buildings constructed before 1945, the average ratio between MRSA and NLTHA results amounts to a factor of about four, as shown in Figure 2. Figure 2 reflects the comparison for clay brick URM built previous and after 1945. There, MRSA method seems to be more competitive for post-1945 URM buildings which usually present rigid concrete floors, lower wall thickness (such as in cavity walls) and less structural modifications during their life span. When separating the applied strengthening measures in Figure 2 according the Groningen Retrofitting Measures Catalogue, GMC (2019), it is evident that the most substantial impact in between MRSA and NLTHA is related to measures to improve out-of-plane, OOP, and in-plane, IP, behaviour. In the GMC (2019) the arrangement of the measures, from L2 (connections) to L6 (foundations) as observed in Figure 2, tend also to order from the most desirable for implementation to the least. There, measures related to L5 and L6 are indicator of structural lateral capacity insufficiency.

Regarding the similarities, a comparable average amount of measures is provided to improve connectivity in between walls and floor/roof, and also applied to improve behaviour of floors and roofs. Both measures related to connections and floors promote the joint behaviour of structural elements; hence, enhancing structural redundancy of the system which comes out into a better global behaviour of the structure.

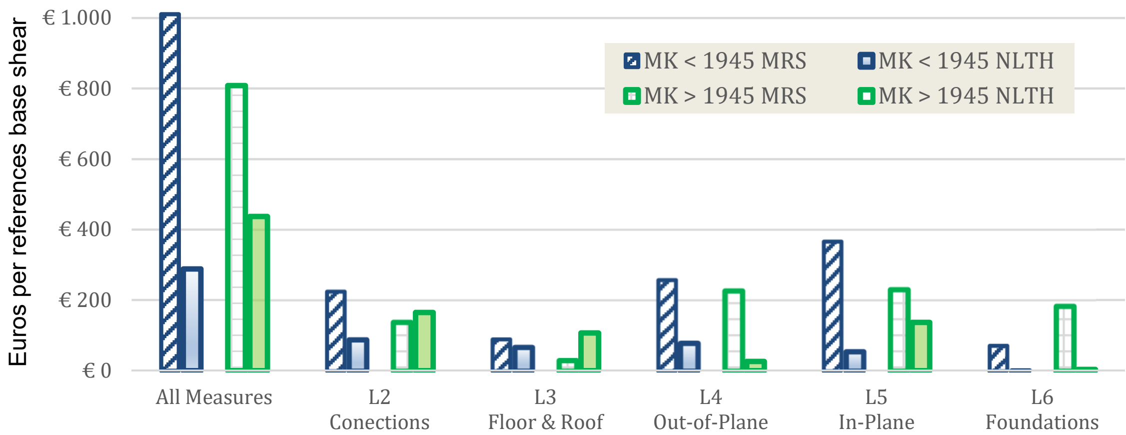


Figure 2. Estimate normalized cost of retrofitting measures for pre-1945 clay masonry (MK) and post-1945 clay masonry by means of MRSA and NLTHA methods

Seismic demand

One of the main differences of NLTHA and MRSA methods is the way seismic actions are applied, either as a signal or from a design spectrum, DS. Within evaluated objects in the VIIA project, surface signals had been calculated as well for MRSA since the NPR 9998 allows a design spectrum reduction up to 20% based on signals analysis. As a result, in both analysis types a set of seven bed-signals had been applied at a depth of 30 meters. For the bed-signal, the acceleration amplitude values of a standarized reference signal had been modified for each method according to recommended NPR 9998 factors (NLTHA bed-signal amplification is 40% higher than MRSA bed-signal).

In order to compare the seismic demand used for both methods, it is relevant to identify the differences that arose. Table 1 summarizes some parameters of the resultant surface waves for five objects, for which both MRSA and NLTHA were conducted. The quantities to characterize the ground motion analysed here are the values of surface PGA and the Arias intensity, I_A. It can be noticed from the table that the horizontal bed-signal used in NLTHA experienced a relevant reduction of the PGA magnitudes due to the non-linear soil behaviour and as a result, they are similar to those calculated for MRSA. In contrast, the delivered I_A is about 50% more in average for the NLTH signal. After obtaining the final DS from the signal analysis for MRSA, a comparison in between the resultant maximum base shears for the objects revealed that the base shear attained in NLTHA is about 10% to 30% higher than the one observed in MRSA. All these observations suggest that differences in retrofitting measures between the methods were provoked by considerations related to the object modelling protocols, the non-linear behaviour of the structure and the NPR 9998 compliance criteria, rather than the applied seismic intensity.

			Signal MRS				Signal NLTH			
	V_s soil (m/s)	$A_{g;ref}$ (%g)	I_A x (m/s)	Surface PGA x (m/s^2)	Surface PGA y (m/s^2)	Surface PGA z (m/s^2)	I_A x (m/s)	Surface PGA x (m/s^2)	Surface PGA y (m/s^2)	Surface PGA z (m/s^2)
Object A	155	0,29	0,83	2,3	2,4	4,1	1,27	2,4	2,5	6,9
Object B	167	0,27	1,09	3,3	3,4	3,8	1,75	3,7	3,8	6,4
Object C	234	0,19	1,86	2,0	2,1	2,7	2,42	1,9	2,2	3,8
Object D	161	0,18	1,21	1,8	1,8	2,5	1,92	1,9	2,0	4,3
Object E	177	0,16	1,64	1,8	1,8	2,3	2,42	1,9	2,2	3,8

Table 1. Comparison in between MRSA (used to define the DS) and NLTHA surface waves

Structure Modelling

In structural modelling process, the consequences derived from differences in aspects such as initial conceptualization of structural and non-structural building components, sketching of the model elements and connections in between elements are reflected in the variations of seismic behaviour. In the MRSA process, it has been a common practice to simplify the structural model since elements are to behave only in the linear-elastic range, e.g. to not explicitly model elements that may be considered to fail during the event, not effectively contributing for the capacity. For instance, walls had been modelled without spandrels under the assumption that these are weak elements that will fail. On the other hand, in the NLTHA process objects in the VIIA project are modelled in detail, even including non-structural elements such as outer leaves of cavity walls.

One of the aspects where the differences in modelling are evident is their influence on the natural frequencies and modes. In MRSA, the main vibration modes are of much relevance since acting forces are calculated directly from the design spectrum at each mode according to the mass participation factors. It has been observed, for the five objects mentioned in Table 1, that for the modes with greater mass participation (and similar modal shape) there is a significant difference in the calculated natural frequencies in both methods; where, a more realistic NLTHA model attained significantly higher dominant natural frequencies than the MRSA. Aspects that can be or are expected to be greatly influencing the differences noticed are related with the following:

- interface foundation - terrain,
- geometrical modelling of the walls,
- modelling and characterization of connections (interface in between elements), and
- roof structure modelling and properties.

To observe modelling differences in a simple building configurations, eight URM models, with calcium silicate bricks, are presented in Table 2a and Table2b. There, the variation of the elastic eigenmodes is observed for aspects like the orthogonal coupling of walls, incorporation of spandrels, alternative roof types and the variation of the modulus of elasticity. In the first four models a rigid floor is provided. In contrast, in the last four models the type of floor and roof varies.

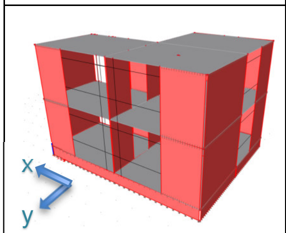
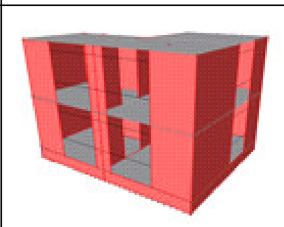
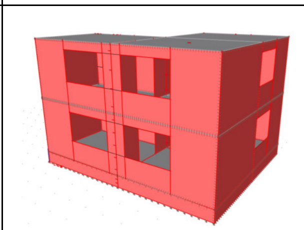
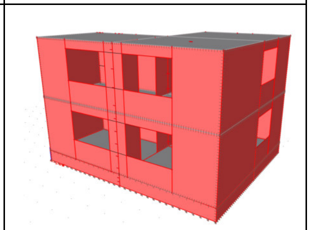
Model 1 *decoupled + only pier/ walls*	**Model 2** *connecting orthogonal walls*	**Model 3** *connecting walls + spandrels*	**Model 4** *connecting walls + spandrels + Double E modulus*
			
Main frequency in x 4,01 Hz (r_1 = 57%)	Main frequency in x 4,66 Hz (r_1 = 56%)	Main frequency in x 7,17 Hz (r_1 = 47%)	Main frequency in x 9,51 Hz (r_1 = 43%)
Main frequency in y 6,49 Hz (r_1 = 55%)	Main frequency in y 7,22 Hz (r_1 = 55%)	Main frequency in y 8,53 Hz (r_1 = 49%)	Main frequency in y 11,37 Hz (r_1 = 24%)

Table 2a. Simple models to indicate the effect of model variation on the fundamental natural frequencies (r = mass participation factor)

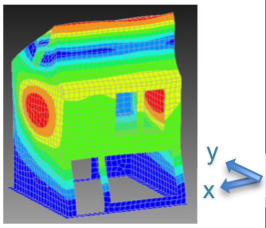
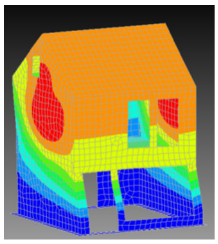
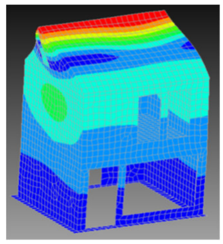
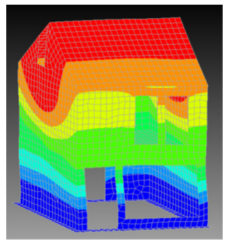
Model 5 *timber floors +* *flexible wooden roof*	**Model 6** *timber floors +* *stiffened wooden roof*	**Model 7** *hollow core slab +* *flexible wooden roof*	**Model 8** *hollow core slab +* *stiffened wooden roof*
			
Main frequency in x 8,65 Hz (r_1 = 41%)	Main frequency in x 7,62 Hz (r_1 = 67%)	Main frequency in x 8,88 Hz (r_1 = 42%)	Main frequency in x 7,66 Hz (r_1 = 77%)

Table 2b. Simple models to indicate the effect of model variation on the fundamental natural frequencies (r = mass participation factor)

The difference in the results from model one with respect to model three, observed in Table 2a, is representative for the modelling conditions observed in MRSA simplifications (model 1) and in NLTHA more realistic models (model 3). The impact of lower natural frequencies in MRSA, to the more realistic ones, is in low-rise URM most possibly an over estimation of seismic loads, hence the total base shear. For the five previously analysed objects in Table 1, the fundamental natural frequencies in between MRSA and NLTHA models vary, in all cases, by about a factor of 2. The modulus of elasticity is incorporated as a parameter related to the material model; where, as permitted in NPR 9998, its value used in the model is an estimated cracked value, half of the initial uncracked quantity.

In Table 2b models are analysed to remark the relevance of an appropriate characterization of roof structures for the structural model. Note that it may not only influence significantly the modal shape, but also the participation factor. In Groningen region, several low-rise URM structures present large roof structures, pointing out the importance of this elements.

Foundation-terrain interface

To avoid or reduce strengthening measures located at the foundations of existing building is desirable since they are expensive and usually complex to perform. In this context, to include in the assessment of existing building the soil-structure interaction (SSI) considerations can bring beneficial effects, both for the foundation elements themselves and to the superstructure. Benefits come by means of natural period lengthening and foundation damping for short-period stiff structures with well interconnected foundation systems founded on soil; with, in contrast, negligible benefits for tall, flexible structures (NIST GCR 12-917-21). Also, in MRSA it will be usually necessary to include several modes to capture the required total mass participation unless soil springs are incorporated into the model (ASCE 7-10). Typically for Linear Static Analysis (LSA) and MRSA, SSI use is limited to apply springs as boundary conditions, representing flexibility of the soil-foundation interface. For these methods, the reduction of the base shear by means of adjusting reduction factors (called R factors or behaviour "*q*" in NPR 9998 context) due to SSI is possibly inappropriate (NIST GCR 12-917-21).

The foundation-terrain interface has been characteristically considered as rigid within VIIA project MRSA objects, with a foundation connected to the ground by means of stiff linear springs, distributed along the shallow foundation or at the location of piles. This assumption is considered suitable to avoid the underestimation (false base isolation) or overestimation (false amplification) of the seismic response of the superstructure and for the case of the presence of disconnected masonry foundations, found in old URM, but may produce unrealistic stress concentration for stiff-connected concrete foundations. The introduction of flexible springs, according to three terrain properties, for a calcium silicate URM with reinforced concrete shallow foundation is observed in table 3. There, to incorporate the soil springs into the model evidenced a reduction of the modal displacements concentration in the upper levels the buildings (as mentioned in ASCE 7-10), evident in the eigenmode shapes, for both stiff and flexible wooden roofs; with a higher reduction for the stiffened roof cases.

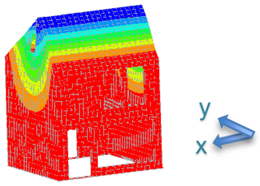
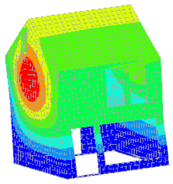
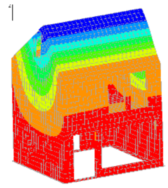
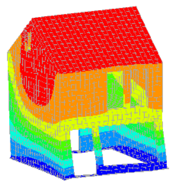
Model A *stiff foundation +* *flexible wooden roof*	**Model B** *stiff foundation +* *stiffened wooden roof*	**Model C** *flexible foundation +* *flexible wooden roof*	**Model D** *flexible foundation +* *stiffened wooden roof*
			
Rigid fixed to the ground eigenmode in x = 4,2 Hz (max r_1 = 26%)	Rigid fixed to the ground eigenmode in x = 8,3 Hz (max r_1 = 71%)	*Stiff Soil in x = 3,8 Hz*	*Stiff Soil in x = 5,3 Hz*
		Dense Sand in x = 3,7 Hz (max r_1 = 49%)	Dense Sand in x = 5,0 Hz (max r_1 = 82%)
		Medium Sand in x = 3,5 Hz	*Medium Sand in x = 4,5 Hz*
Rigid fixed to the ground eigenmode in y = 5,8 Hz (max r_1 = 12%)	Rigid fixed to the ground eigenmode in y = 14,6 Hz (max r_1 = 13%)	Dense Sand in y = 8,3 Hz (max r_1 = 39%)	Dense Sand in y = 7,0 Hz (max r_1 = 78%)

Table 3. Images for the fundamental eigenmodes in x for various spring foundation interface

In Table 3, the vertical subgrade values for various soil types is approximated from typical values recommended from Bowles (Bowles, 1995), the final applied static impedance spring values are estimated according to the foundation characteristics (equations available in NIST GCR 12-917-21 and Miura, 2006). The use of low spring stiffness values, that may be estimated for soft soils, are not recommended since they can lead to major underestimation or overestimation of the seismic demand. Foundation spring values should be calculated with precaution, according a geotechnical site study.

For the case of pile foundations, the fixation spring values typically used in VIIA project for the vertical direction present not as large a difference in magnitude as the one observed for the shallow foundations. Nevertheless, the same observation seems not necessarily valid for horizontal springs values, where the spring value simplifications are usually more influenced by the terrain than the pile characteristics. In Figure 3, the envelope variation (of eight seismic load combinations) in the axial loads acting on the well-connected concrete pile elements is shown (compression(+), tension(-)). The total pile population of the building is of 234 elements. The dispersion observed in the figure evidences higher load concentrations for some piles when stiff fixing spring values (HS) are used, meanwhile when more realistic spring values (RS) are used this dispersion reduces. Also from the figure, it can be noticed that under the static self-weight combination, the axial load distribution dispersion is much lower, making evident the significant impact of seismic loads on this pile foundation for this structure.

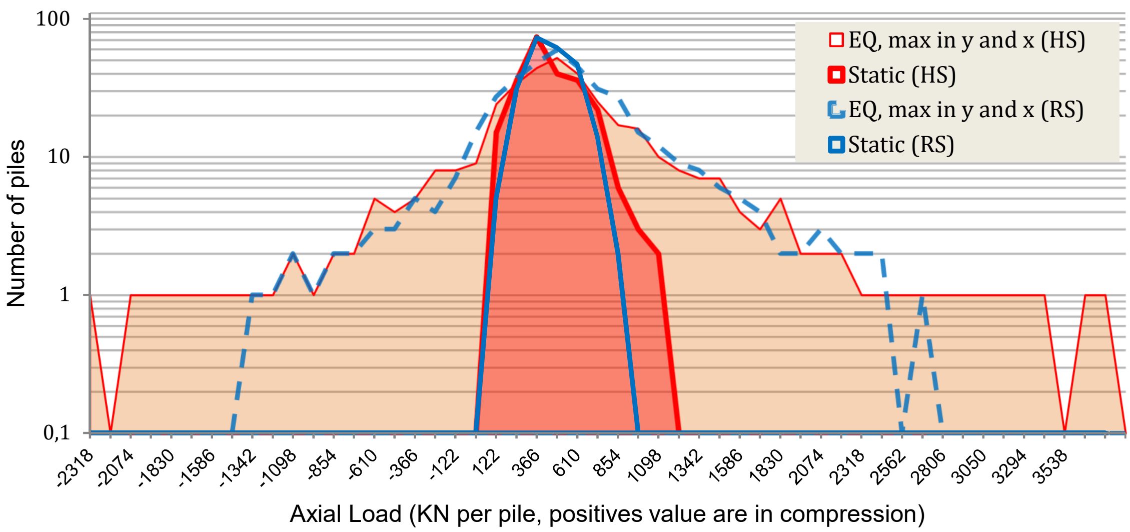


Figure 3. Maximum axial load on pile population (234 piles) for all earthquake combinations (envelope) and the self-weight for a vertical stiffness K_v=500 MN/m (RS) & K_v=2500 MN/m (HS)

Regarding the impact of horizontal springs for pile foundations, it has also been observed, for the same object analysed in Figure 3, that the high stiffness values may contribute additionally to torsion of the structure, this due to pile concentration on one side of the building (vertical irregular building). The torsional effects can be observed in Figure 4 by the difference in direction of the base reactions, where the seismic load combination applied is of 100% of the seismic load in the +Y direction and 30% of the seismic load in the +X direction. Similarly to shallow foundations, horizontal stiffness has also an impact on the natural frequencies, with dominant modes changes from 1,9 Hz to 2,2 Hz activating the five storey section of the structure and from 5,2 Hz to 7,0 Hz at the two storey section. This supports the previous affirmation that SSI considerations in MRSA are more significant for low rise structures.

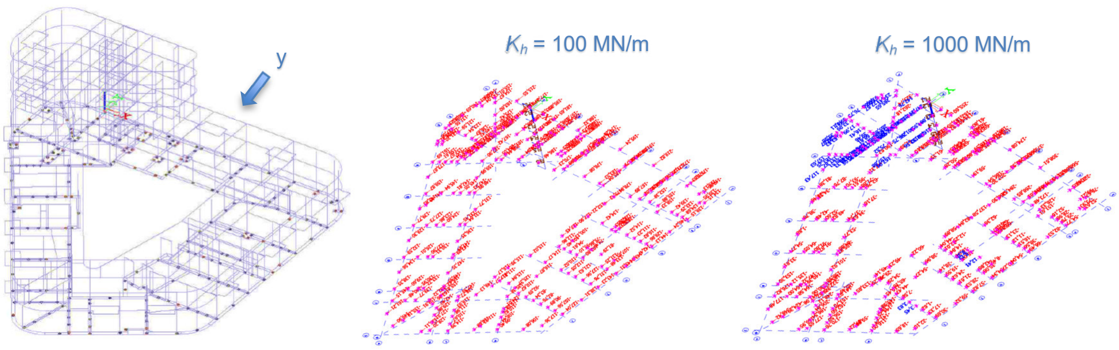


Figure 4. Lateral load in "Y" for load combination (100% +Y 30% +X) with horizontal stiffness values K_h=100 MN/m and K_h=1000 MN/m. In blue are loads in -Y, and in red loads in +Y

In-Plane assessment

A major variation had been observed from Figure 1 for the average cost comparison in between MRSA and NLTH methods. In particular, a greater difference from the database is observed for clay URM constructed before 1945, where the presence of timber floors is common. This can be expected since MRSA is a force-based method, which yields a more conservative assessment as compared to displacement-based methods.

Typically, as mentioned in the NPR 9998, a behaviour factor of 1,5 is recommended for URM according to the NEN-EN 1996 -1-1. This might be conservative, in particular when sufficient connectivity existed or is provided in between walls and floors. The later suggests that the global ductility of URM buildings in Groningen should be further investigated and behaviour factors, q, calibrated. The NPR 9998 allows q factor calibrations by non-linear methods, such as non-linear pushover analysis or the increment of the q factor up to 2,0 if the masonry comply with the requirements mention in NEN-EN 1998-1 and according to a α factor.

The α factor is no more than the ratio in between the acting overburden stress and the characteristic compression resistance of the masonry. This ratio is a parameter of importance to identify brittle failure modes as it can be observed from the equations for the lateral shear capacity determination presented in annex G in the 2017 draft NPR 9998:2017. By means of these equations, failure mode zones are presented in Figure 5 for pre-1945 clay URM.

To check of piers/walls according to pre-elaborated images such as Figure 5, is a way to assess if elements are expected to behave according a certain failure mode; so that, the use of a higher q factor, related to higher global ductile behavior, is justified by the local behavior of most elements. When doing this, special care must be observed on local elements not failing in a ductile manner, since they must be discarded to represent a structural weakness that may control the global behavior. Also, it may be observed from Figure 5 that walls with low α values are expected mostly to present a rocking behaviour. This condition may be observed for many low-rise URM.

Finally, different from the feasibility of MRSA to analyse a particular structure, it is important to mention its intrinsic dependence on the seismic demand values for the assessment (unity check). Under the circumstance of a varying seismic demand as a function of the gas production, the possibility of straightforward reassessment of structures is not the case for MRSA, where renewed seismic actions will need to be calculated. Neither a better situation is found for the deterministic seismic signal dependent NLTHA. Here, NLPO evaluations results are usually easy to adjust to a new seismic scenario, although not all structures are suitable to be assessed by means of this method.

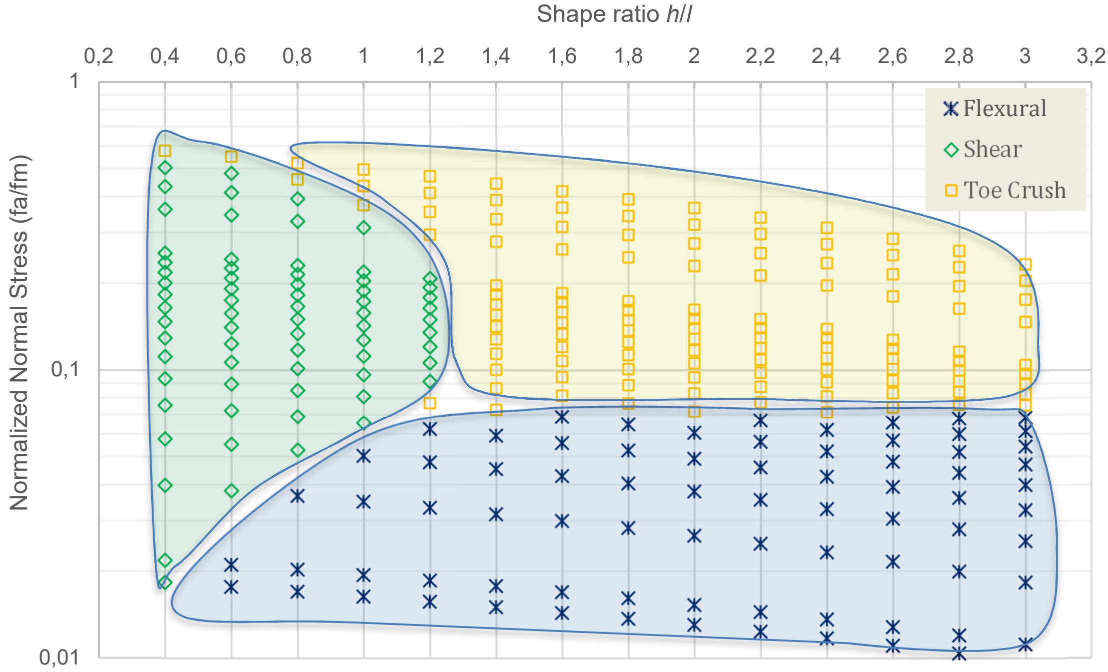


Figure 5. Expected failure mode of pre-1945 clay masonry in Groningen according to the wall/pier geometry and overburden stress (h= height, l = length, fa = overburden stress and fm = masonry compression resistance stress)

Out-of-Plane assessment

The out-of-plane assessment from MRSA outputs is usually accomplished by evaluating bending stresses to the flexural tensile strength in the linear range (linear elastic analysis LEA). For URM material walls, this brings the necessity to retrofit several walls in the out-of-plane (OOP) direction; even for a low seismic demand and the use of a behavior factor of 2 for well-connected inter-story walls. This aspect is also evidenced after the inspection of Figure 1, when comparing the MRSA and NLTHA results in terms of cost of retrofitting measures. The influence of certain modelling conditions for LEA assessment, such as connecting walls with their orthogonal walls and the introduction of a non-fixed terrain-foundation interface, are beneficial for the reduction of the seismic driven stresses on the walls. Regarding the last condition, flexible spring interfaces at the foundation reduce the modal local displacements associated with the individual vibration of structural components in the out-of-plane direction.

Different from the LEA, the non-linear kinematic analysis, NLKA, described in the annex H of the recent NPR 9998:2018, provides a suitable method to assess the OOP, identifying compliance or not of walls with aid of acting overburden forces that can be set to the lower overburden value from all seismic load combinations. The NLKA method is based on analysis of a rocking mechanism representing the failure mode that is observed when 1-way spanning walls fail OOP by formation of local cracks on top, bottom and mid-height of the wall. Stability of this rocking mechanism is analyzed by means of virtual work. A Comparison in between LEA and NLKA results for walls well connected are shown in Table 4 (NLKA according to NTC-2009).

Wall thickness (mm)	**Method**	**Overburden load P** (W = self-weight of the wall)		
		P=0	P=2W	P=4W
t = 110	LEA	0,20	0,34	0,48
	NLKA	0,90	2,70	4,50
t = 220	LEA	0,80	1,36	1,93
	NLKA	3,60	10,80	18,01

Table 4. Comparison in between LEA (connected only at the bottom and top) and NLKA equivalent moment capacity (in kNm/m) for a well-connected 3,5 m height URM wall

Conclusions

Prior to the gas extraction activities, the region of Groningen has been considered as a very low seismicity area; in fact, located in the zone with the lowest expected seismic intensity for The Netherlands. This implies several challenges; in particular, for the safety assessment of existing URM structures, which corresponds to about 85% percent of the building stock in the threatened area and are considered highly vulnerable to earthquakes. Most structures consist of low-rise buildings, which typically in seismic active regions would be analysed by means of simple LSA but, due to the commonly present structural irregularities, are in many occasions only possible to be analysed by means of MRSA or a higher degree non-linear method.

This article had discussed several optimization actions for MRSA method based on the analysis of database results in terms of the costs of retrofitting measures according to the MRSA and NLTHA results and the experience attained from the VIIA project after conducting MRSA method, as described in the NPR 9998:2015, for several structures for the Groningen region. An overview of the MRSA and NLTHA results, in terms of strengthening cost, allows a pragmatic comparison in between measures for different retrofitting goals or elements. The strengthening goals are defined according to the GMC arrangement of the seismic retrofit measures; where the categories are: L2 (connections), L3 (floors & roof), L4 (Out-of-Plane), L5 (In-Plane) and L6 (foundations), as observed in Figure 2.

Observations related to the use of MRSA in the region of Groningen had been overviewed. In particular, the method is expected to give better results for areas with low to middle seismic hazard levels and small to middle size structures, as already commented for Figure 1. For high hazard threat areas, MRSA may result in too many L5 and L6 measures, making it possibly necessary, from an economical point of view, to use non-linear methods to assess the structure. It has been also noticed, that most probably differences in between MRSA and NLTHA within VIIA project assessments come from the resistance of the structures to earthquakes and the assessment criteria for each method rather than a higher seismic action applied in MRSA.

The remarks presented highlighted on modelling considerations and assumptions/methods thought to bring improvements to the out-of-plane and in-plane assessment procedures for MRSA, since it is economically and technically convenient to focus on the reduction of L4, L5 and L6 retrofit measures. The observations can be summarized in the following:

- To model the interface foundation-terrain usually brings advantages in terms of reducing the number of natural frequencies to be calculated to arrive to the required mass participation, a smoother distribution of seismic actions throughout the structure and foundation, and a reduction of the modal displacements in the upper levels. The reduction of base shear due to foundation damping is not recommended for MRSA, where actually a minor increment is expected from natural periods lengthening. Rigid fixed connections are recommended to be used for buildings with masonry foundations.
- Aspects related for the geometrical modelling of the walls, modelling/characterization of connections (interface in between elements) and roof structure modelling/properties may present a major impact in the natural periods. This impact may be significant, evidenced for instance in a reduction of the base shear from shortening of the model natural periods.
- The use of NLKA method presents a major advantage for the OOP assessment. Care in use of the NLKA must be observed when determining adequate overburden loads and realistic boundary conditions, not to overestimate the capacity.
- A higher value of the behaviour factor than 1,5 may be used for the assessment of URM, as commented by the NPR 9998. Observations about a conscientious use of higher *q* values according to the expected failure mode of the piers/walls have been commented.

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