Adaptive force-based procedure for the seismic design of masonry buildings

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Abstract

Masonry is one of the oldest building concepts and offers great sustainability features. However, the potential of masonry for the construction of new buildings is severely hindered by the over conservatism of current masonry design practice. This paper presents an alternative, practice-oriented procedure, named adaptive force-based procedure, for the seismic design of masonry buildings. The developed procedure implements nonlinear static (pushover) analysis to individually estimate the values of the overstrength ratio for each building, and hence, to lower uncertainties associated with the behaviour factor of masonry buildings. This enables the adaptive force-based procedure to provide less conservative designs without requiring a fundamental change in the current design practice.

1. Introduction

There is an intense pressure on the construction industry, as a consumer of large quantities of resources as well as a main source of solid waste generation and greenhouse gas emissions, to include sustainability features in the construction of new structures and the preservation of existing ones. Masonry is one the most sustainable building materials; it offers various economic, environmental and social benefits like ease and speed of construction, superior sound and thermal insulation, fire resistance, durability, low maintenance, eco-friendliness, aesthetics and most importantly, short- and long-term cost efficiency [1]. However, possibly due to the common fallacy of unsuitability of structural masonry for construction in seismic areas, the need for research on the seismic design of new masonry structures has not been properly appreciated [2]. Hence, the current masonry design practice is too conservative, particularly concerning the seismic design. As a result, the share of masonry structures in the construction market is limited to some low-rise buildings in areas of low seismicity. Therefore, to effectively deploy the sustainability of masonry, the development of appropriate approaches for the seismic design of masonry structures is inevitable. To meet this need, a research project was initiated in Switzerland in a collaboration between ETH Zurich, Promur Association [3] and Cubus AG [4]. The main goal of the project was to contribute into the exploitation of the sustainability of masonry by providing the engineering community with an advanced but still practical procedure for the seismic design of masonry structures. This paper presents the developed procedure named "adaptive force-based" procedure.

2. Current approaches for the seismic design of masonry structures

In general, two approaches are recognised by codes for the seismic design of structures: the force-based (FB) and the displacement-based (DB) approaches. The FB approach is based on linear analysis and interprets the seismic demand and capacity in force terms. Figure 1 shows schematically the capacity curve, i.e. the base shear vs. the top displacement (the displacement at control point), of a building and a bilinear idealisation of it. According to the FB approach, the building satisfies the strength requirement at the ultimate limit state if:

$$F_{el} \geq F_d/q \qquad \text{Eq. 1}$$

where F_{el} is the base shear corresponding to the first attainment of shear resistance in a structural element, F_d is the base shear demand and q is the behaviour factor of the structure.

The behaviour factor (q) plays a fundamental role in the FB design approach allowing the benefits offered by nonlinear response of structures to be used. The value of q is obviously a crucial choice in the FB approach. Most current codes, e.g., Eurocode 8 [5], ASCE 7 [6] and Swiss masonry standard (SIA 266) [7], suggest a range of values between 1.5 and 2.5 for the behaviour factor of unreinforced masonry (URM) buildings, keeping however the lower limit, i.e. 1.5, as the recommended value. In practice, requiring such overconservative (low) values for the behaviour factor severely limits the possibility of construction with masonry even in areas of low seismicity. For example, in a numerical study carried out by Morandi [8], it was shown that with a q of 1.5 or even 2, it is practically impossible to satisfy the ultimate limit state resistance requirements of Eurocode 8 for any configurations of two- or three-storey URM buildings for $a_g.S$ (a_g and S are the design ground acceleration on rock and the soil factor) greater than 0.1g, and in many cases even for $a_g.S$=0.05g. This is surprisingly inconsistent with Eurocode 8 rules for "simple masonry buildings" that allow for instance a two-storey URM building to be constructed in an area with $a_g.S$=0.15g; see [5]. This is also in great contradiction with experiences of past earthquakes, experimental findings and numerical evidence that are based on nonlinear analysis: According to Magenes [2], by comparing hazard maps of Italy expressed in terms of a_g and macroseismic intensity scales, both defined for the same return period, it can be inferred that on the base of the observation of past earthquakes and of the safety levels accepted by current seismic codes, the performance of engineered low-rise URM buildings should be considered adequate for areas with a_g up to 0.3g (most of Italy). In another study, Lourenço et al. [9] showed that based on nonlinear static (pushover) analysis, low-rise URM buildings can be constructed in most of Portugal, with only restrictions in areas with a_g greater than 0.20g. The post-earthquake inspections conducted after the 2012 Emilia earthquake sequence seems to confirm the outcome of the abovementioned studies; In fact, no significant damage in structural and non-structural elements was found in the great majority of engineered masonry buildings even in areas with a_g as large as 0.25-0.30g [10].

To explain such a contradictory panorama, careful attention should be paid to the definition of the behaviour factor. According to the FB approach, the ultimate state of a structure corresponds to the first attainment of shear resistance in a structural element. Hence, the definition of the behaviour factor would be (see Figure 1):

$$q = \frac{F_{el,max}}{F_{el}} = \frac{F_{el,max}}{F_{max}} \cdot \frac{F_{max}}{F_{el}} = q_0 \cdot OSR \qquad \text{Eq. 2}$$

where $F_{el,max}$ is the base shear of the equivalent linear system of the structure (not to be confused with F_d) and F_{max} is the shear resistance of the structure. Therefore, q is the product of the basic value of the behaviour factor (q_0) and the overstrength ratio (*OSR*).

In URM buildings, q_0 shows a minor variation within different buildings (usually ranges from 1 to 2) while the range of *OSR* values is very wide (usually between 1.5 to 6) [11–13]—the values of *OSR* is often very different even for the two main orthogonal directions of the same building [14]. Such a variation in *OSR* values is mainly due to the great dependency of *OSR* to the structural configuration, i.e. the building layout. Considering the great variation in the values of overstrength ratio, it is practically impossible for codes to prescribe less conservative, but still sufficiently safe values for the behaviour factor of URM buildings; see also [14]. In fact, current codes mostly ignore the contribution of *OSR* in the evaluation of the behaviour factor of URM buildings, which explains their choice of q=1.5-2.5. This trend can be also observed in most pioneer works on the behaviour factor of URM buildings; see e.g., [15–17].

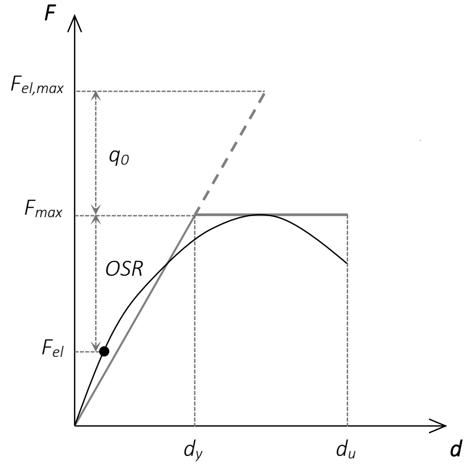

Figure 1: Definition of the behaviour factor

Despite the recent attempts to improve the rationality of the FB approach for the design of URM buildings (either through considering prescribed overstrength ratio values [12] or through alternative procedures like unrestricted force redistribution and capacity-based design [11]), the limitations of linear models in the seismic analysis of masonry buildings clearly emphasize on the necessity for approaches that are based on nonlinear structural analysis. Based on the positive experience gathered during the recent past in developing the basis for the DB design of structures, it appears that the most feasible approach to enhance the rationality for the design of masonry structures is to apply the same basis. The DB approach overcomes the problems of the FB approach by implementing nonlinear analysis and interpreting the seismic demand and capacity in displacement terms. Unlike the FB approach, the DB approach directly considers the benefits offered by the nonlinear response of structures, and hence, eliminates the need for the behaviour factor (q). A more consistent representation of the seismic demand as well as of the seismic capacity leads to more reliable and at the same time more economical designs.

While it is well known that the DB approach is conceptually a better way to implement the seismic design concepts, its application for the seismic design of masonry structures is severely hindered by the unfamiliarity of the engineering community with this relatively new concept, the great difficulties in the modelling and interpreting the post-peak response of masonry structures and the pale presence of the DB approach in current codes—Eurocode 8 [5] and SIA 266 [7] explicitly recognise the DB approach for the seismic design of masonry structures, but provide no proper guidance for its implementation. As a result, the current masonry design practice is still dominated by the traditional FB approach, which is known to be overconservative. The following section proposes an alternative procedure for the seismic design of masonry structures that offers the benefits of both the FB and the DB approach while avoids their problems.

3. Adaptive force-based procedure

The adaptive force-based (AFB) procedure falls essentially under the category of the FB approach as it interprets the seismic demand and capacity in force terms. However, unlike conventional FB procedures, the AFB procedure implements nonlinear static (pushover) analysis, and therefore, has direct access to the shear resistance of structures (F_{max}); see Figure 1—note that conventional FB procedures use linear analysis allowing access only to F_{el}. Hence, recalling the definition of the behaviour factor, Eq. 1 can be rewritten as:

$$F_{el} \geq \frac{F_d}{q} \rightarrow F_{el} \geq \frac{F_d}{q_0 \cdot OSR} \rightarrow F_{el} \cdot OSR \geq \frac{F_d}{q_0} \rightarrow F_{max} \geq F_d / q_0 \qquad \text{Eq. 3}$$

In other words, in the AFB procedure, the compliance of a structure with the strength requirement of the ultimate limit state is verified if the shear resistance of the structure obtained from pushover analysis (F_{max}) is greater than the base shear demand (F_d) divided by the basic value of the behaviour factor (q_0) of the structure. As can be seen from Eq. 3, in the AFB procedure, the contribution of the *OSR,* which is the problematic part of the behaviour factor, is directly considered in the shear resistance of structure (F_{max}) and only the value of q_0 is needed as input.

The implementation of pushover analysis gives the AFB procedure the privilege to estimate the *OSR* value individually for each building—this explains the idea behind its name. By eliminating the uncertainties associated with the *OSR*, the AFB procedure provides designs that are much less conservative than those provided by conventional FB procedures relying on overconservative behaviour factors prescribed by codes. It should be emphasized again that due to the great variation in the values of *OSR*, it is practically impossible to prescribe less conservative values for the behaviour factor of masonry buildings.

In comparison with DB procedures, the AFB procedure offers two main advantages: First, it works in the force domain that is more familiar to the engineering community, and secondly, unlike DB procedures, it requires only the pre-peak section of the capacity curve of structures (up to F_{max}), and therefore, avoids the problems associated with the modelling of the post-peak response of masonry structures. Furthermore, in the AFB procedure, the pushover analyses can be terminated as soon as the ratio between the applied base shear and the base shear demand (F/F_d) reaches one (or any other value if required) to save on computation time. Nevertheless, it should be noted that the AFB procedure remains still slightly more conservative than DB procedures because of the uncertainties associated with the value of q_0. In Summary, the AFB procedure offers a simple, practical and reasonably conservative solution for the seismic design of masonry buildings without demanding a dramatic change in the current masonry design practice. As mentioned before, the AFB procedure requires the value of q_0 as input. In the absence of code-prescribed values, the following section gives some recommendations regarding the value of q_0 for URM buildings albeit with a focus on the current Swiss masonry practice.

4. Basic value of the behaviour factor of URM buildings

4.1. Building inventory

In order to investigate the basic value of the behaviour factor (q_0) for contemporary URM Buildings, 16 buildings from three to seven stories were considered; see Table 1. The buildings consist of storey-high URM walls without openings and in-plane rigid reinforced concrete slabs, which is a common practice for the construction of URM buildings in Europe. Some buildings have also reinforced concrete columns, but they do not contribute into the lateral resistance of the structure. The buildings are regular in plan and elevation—in each building, all the stories have the same height (3 m) and the same plan; see Annex A for the building plans. The masonry walls are single-leaf walls made of standard masonry according to SIA 266 [7] with the mechanical properties given in Table 2. In each building, all the walls have the same thickness. All the slabs are 20 cm thick. Furthermore, it was assumed that the buildings provide effective connections between intersecting walls as well as between walls and slabs.

Table 1: Building inventory

Building	No. of stories	Storey Plan	Walls' thickness (cm)
B1_3_20	3	P1	20
B1_3_15	3	P1	15
B1_5_20	5	P1	20
B1_5_30	5	P1	30
B1_7_30	7	P1	30
B2_3_20	3	P2	20
B2_5_20	5	P2	20
B3_3_20	3	P3	20
B3_5_20	5	P3	20
B4_3_20	3	P4	20
B4_5_20	5	P4	20
B5_3_20	3	P5	20
B5_3_15	3	P5	15
B5_5_20	5	P5	20
B6_3_15	3	P6	15
B6_5_20	5	P6	20

Table 2: Mechanical properties of masonry

Property	Symbol	Value
Dimensioning value of the compressive strength of masonry normal to bed joints	f_{xd}	3.50 MPa
Dimensioning value of the compressive strength of masonry normal to head joints	f_{yd}	1.60 MPa
Dimensioning value of the coefficient of internal friction in the bed joints	μ_d	0.60
Characteristic value of elastic modulus for masonry loaded normal to bed joints	E_{xk}	7.00 GPa
Characteristic value of the shear modulus of masonry	G_k	2.80 GPa

4.2. Structural modelling and analysis

It was assumed that local brittle failure modes, which are usually associated with the out-of-plane response of the walls, were prevented, and therefore, a global box behaviour governed by the in-plane response of the walls would develop. The buildings were then modelled as three-dimensional equivalent frames. The suitability of the equivalent frame approach for the analysis of the global seismic response of URM buildings has been verified in several studies; see e.g., [18–20]. The walls were assumed to have linear elastic-perfectly plastic response with limited displacement capacity while the slabs were supposed to remain linear elastic. In order to define the bilinear response of URM walls, three parameters were needed: the effective stiffness (K_{eff}), the ultimate shear resistance (V_u) and the ultimate drift ratio capacity (δ_u); see Figure 2. In this study, as proposed by most codes, the effective stiffness was taken as one half of the uncracked stiffness calculated based on the characteristic values of elastic and shear moduli according to the elastic beam theory incorporating the shear deformation. The ultimate shear resistance (V_u) was taken as the dimensioning value of the shear resistance calculated according to SIA 266 [7]. Finally, the ultimate drift ratio capacity of the walls was estimated according to the empirical model proposed by document SIA D0257 [21]:

$$\delta_u = \delta_0 \left(1 - \frac{N_x}{l_w t_w f_{xd}}\right) \frac{h_v}{h_w} \qquad \text{Eq. 4}$$

where h_w, l_w, t_w and h_v are respectively the height, length, thickness and the shear span of the wall, N_x is the axial force of the wall at the stage of attainment of the ultimate shear resistance, f_{xd} is the dimensioning value of the compressive strength of masonry normal to bed joints and δ_0 is the base value of the drift ratio capacity. δ_0 represents the effect of the constituent materials of masonry, and its value in this study was taken as 0.4%, which corresponds to the design value of drift ratio capacity of standard masonry according to SIA 266 [7].

The above mentioned model was developed by Salmanpour et al. [22] and has the advantage of being independent of the failure mode making its implementation straightforward. Furthermore, it considers the influences of constituent materials, pre-compression level and boundary conditions on the displacement capacity of URM walls. For a through discussion on the available models for the displacement capacity of URM walls see [23,24].

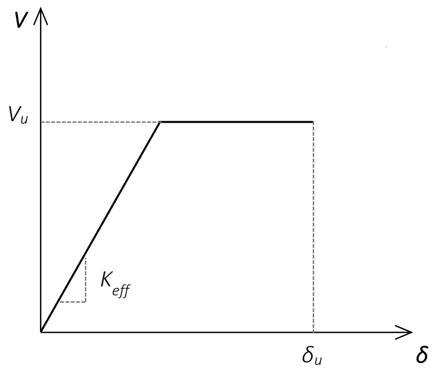

Figure 2: Force-drift ratio response of URM walls

In order to simulate the nonlinear response of URM walls, concentrated plastic hinges were implemented. The capacity curves of the buildings were obtained through pushover analysis. Considering two vertical distributions of the lateral loads, i.e. uniform and modal patterns, and the torsional effects due to accidental eccentricity, 16 pushover analyses for each building were performed. The ultimate limit state of the buildings was assumed to be attained as soon as the drift ratio of a wall reached its ultimate drift ratio capacity (according to Eq. 4) or a wall was no longer able to carry its axial load. It should be mentioned that this assumption about the ultimate limit state of URM buildings could be conservative since it does not consider the possibility of redistribution of vertical loads. However, it significantly facilitates efficient pushover analysis of URM buildings. The buildings were modelled and analysed using MURUS-P computer code; for more information see [25]. The obtained capacity curves are presented in Annex B.

4.3. Methodology for the estimation of q_0

The capacity curves obtained from pushover analysis were first idealised by linear elastic-perfectly plastic curves according to the procedure proposed by Eurocode 8 [5]. As shown in Figure 3 (for Y direction of Building B3-3-20), the yield force and the ultimate displacement of the idealised curve were taken equal to the maximum force (shear resistance) and the ultimate displacement of the actual capacity curve. Afterwards, the initial stiffness of the idealised curve was determined by equating the areas under the actual and idealised curves to provide energy equality. The parameters of the idealised capacity curves are given in Annex C.

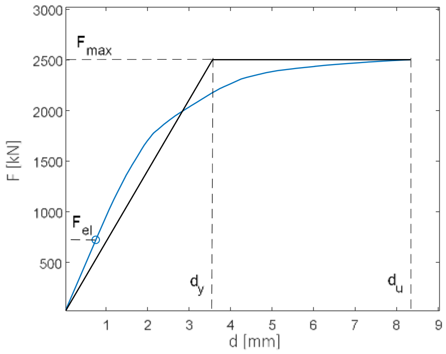

Figure 3: Idealised capacity curve for Y direction of Building B3-3-20

Several different procedures can be found in the literature for the estimation of q_0. Veletsos and Newmark [26] noticed that for structures of medium to large periods, the ultimate displacements of elastic and inelastic systems were approximately the same. According to this empirical statement known as "equal displacement principle", the value of q_0 equals the ultimate ductility of the bilinear system (μ):

$$q_0 = \mu = d_u/d_y \quad \text{Eq. 5}$$

The validity of the equal displacement principle for medium- and long-period structures has been confirmed by numerous studies; see e.g., [27]. Eurocode 8 [5] proposes an adjustment to the equal displacement principle for short-period structures based on a simplification of N2 method [28]:

$$q_0 = \begin{cases} (\mu - 1) \cdot \dfrac{T}{T_c} + 1, & T < T_c \\ \mu, & T \geq T_c \end{cases} \quad \text{Eq. 6}$$

where T is the period of the idealised equivalent SDOF system, and T_c is the upper period limit of the constant spectral acceleration branch. According to Eq. 6, q_0 varies linearly from 1 for the period of zero to μ for the period of T_c and remains constant afterwards (equal displacement principle). The value of T_c depends on the ground type and the seismicity level and is usually between 0.25 and 0.4 second. Another procedure for short period structures is the so-called "equal energy principle" that estimates q_0 using the energy conservation assumption as:

$$q_0 = \sqrt{2\mu - 1} \quad \text{Eq. 7}$$

The equal energy principle has been often used in the past for the evaluation of q_0 of masonry structures, e.g., [16,17,29], and has the advantage of being independent of the elastic response spectrum. Annex C gives the q_0 values obtained from the abovementioned procedures as well as the values of *OSR* estimated as:

$$OSR = F_{max}/F_{el} \qquad \text{Eq. 8}$$

For Eurocode 8 procedure, the values of T_c were taken as those recommended by Eurocode 8 for the Type 1 response spectra, which are consistent with the values recommended by the Swiss code "Actions on Structures" (SIA 261) [30]; see Table 3. For the type 2 response spectra, which are recommended for areas of low to moderate seismicity, Eurocode 8 procedure coincide with the equal displacement principle.

Table 3: Values of Tc (s)

Spectrum	Ground Type				
	A	B	C	D	E
Type 1	0.4	0.5	0.6	0.8	0.5
Type 2	0.25	0.25	0.25	0.3	0.25

Table 4 presents a summary of the obtained q_0 and *OSR* values categorised based on the number of stories of the buildings. The characteristic values (charcs) are the values below which it is unlikely that more than 5% of results will fall, and are estimated based on the assumption of a normal distribution for the variables as:

$$\text{charcs} = \text{mean} - 1.64 \cdot \text{stdev} \qquad \text{Eq. 9}$$

Table 4: Summary of q_0 and *OSR* values

	$q_{0,edp}$ (1)$=\mu$	$q_{o,eep}$ (2)	$q_{0,EC8}$, Ground Type				*OSR*
			A	B, E	C	D	
3-Storey Buildings							
No. of data	16	16	16	16	16	16	16
min	1.36	1.31	1.24	1.20	1.16	1.12	1.64
max	2.84	2.16	2.07	1.85	1.71	1.53	5.48
mean	2.08	1.77	1.64	1.51	1.42	1.32	2.92
stdev(3)	0.37	0.21	0.20	0.16	0.13	0.10	1.01
charcs(4)	1.47	1.42	1.31	1.25	1.21	1.15	1.27
5-Storey Buildings							
No. of data	14	14	14	14	14	14	14
min	1.12	1.11	1.13	1.10	1.08	1.06	1.66
max	1.67	1.53	1.56	1.45	1.37	1.28	5.99
mean	1.40	1.34	1.38	1.30	1.25	1.19	3.20
stdev(3)	0.16	0.12	0.13	0.10	0.08	0.06	1.33
charcs(4)	1.15	1.15	1.17	1.14	1.12	1.09	1.01
7-Storey Building							
No. of data	2	2	2	2	2	2	2
min	1.20	1.18	1.20	1.18	1.15	1.11	1.81
max	1.31	1.27	1.31	1.30	1.25	1.18	2.32
mean	1.25	1.23	1.26	1.24	1.20	1.15	2.07
stdev(3)	0.08	0.06	0.08	0.08	0.07	0.05	0.36
charcs(4)	1.13	1.12	1.13	1.10	1.09	1.06	1.47

(1) Equal displacement principle
(2) Equal Energy principle
(3) Standard deviation
(4) Characteristic value

4.4. Proposed values of q_0 for URM buildings

It can be seen from Table 4 that the q_0 values obtained from Eurocode 8 procedure are overall slightly smaller than those obtained from the equal displacement and the equal energy principles. It is however well known that Eurocode 8 procedure can be too conservative in regard to q_0 values of short-period structures mainly because it oversimplifies the lower limit of the period range where the equal displacement principle is valid (by taking it equal to T_c). This transition period is a complex parameter influenced by the ultimate ductility (μ). As shown by Miranda [31], the equal displacement principle is valid from approximately T=0.2 s for μ=1.5 and from T=1.2 s for μ=1.5. Hence, for the buildings considered in this study, the equal displacement principle is reasonably valid; for the building periods, see Annex C: Table C1. Furthermore, as mentioned before, for areas of low to moderate seismicity (Type 2 spectra), Eurocode 8 procedure coincide with the equal displacement principle. The equal energy principle provides q_0 values that are in good agreement with those obtained from the equal displacement principal, but still more conservative. Furthermore, the energy conservation assumption is not supported by a sound scientific background [27]. Hence, given the fact that the ultimate ductility of the buildings were already estimated based on a conservative assumption about the ultimate limit state of the buildings (see Section 4.2) and the design value of drift ratio capacity of the walls, the equal displacement principle was taken as the representative methodology to propose q_0 values for URM buildings.

Considering the characteristic values of $q_{0,edp}$, the following values are recommended for the seismic design of URM buildings that are regular both in plan and elevation:

- for buildings up to and including 3 stories: q_0=1.45
- for building with 4 and 5 stories: q_0=1.15
- for building with more than 5 stories: q_0=1

For non-regular buildings, without further investigation, it is recommended to take q_0 as one.

Nevertheless, it should be mentioned that the importance of q_0 for URM structures is inferior to that of *OSR* so that the AFB procedure can still provide much less conservative designs than conventional FB procedure even with q_0=1. For example, Building B1-3-20 (see Table 1) does not confirm to the strength requirements anywhere in Switzerland according to the lateral force method and a q of 1.5 or even 2. However, using the AFB procedure, the building would satisfy the strength requirements in zones Z1 (a_g=0.06g) and Z2 (a_g=0.10g) [30] with q_0 of 1 and 1.45, respectively—the ground type of B (S=1.2) and importance factor of 1 were considered.

5. Discussion

As can be seen from Table 4, the standard deviation of *OSR* is much higher than that of q_0 for all the categories and independent of the methodology used for the estimation of q_0. Note that in the case of 5-story buildings, the standard deviation of *OSR* is so high that reduce the characteristic value of *OSR* down to one. This confirms the difficulty of providing less conservative behaviour factors (q) for conventional FB procedures as discussed in Section 2. The mean value of *OSR* is also higher than that of q_0 highlighting the efficiency of the AFB procedure in which the *OSR* is estimated individually for each building. It should be noted that in some cases the *OSR* was found to be more than three times larger than q_0; see e.g., buildings B6-3-15, B5-5-20 and B6-5-20 in Annex C: Table C1.

Figure 4 shows the effect of the number of stories (building height) on q_0. In general, by increasing the number of stories, while the lateral stiffness of buildings decreases, the pre-compression level of walls, and therefore, the shear resistance of buildings increases. Accordingly, both d_y and d_u increase. However, since the increase in d_u is smaller than that in d_y, the ultimate ductility (μ), and hence, q_0 decrease; see e.g., buildings B1-3-20 and B1-5-20 in Annex C: Table C1. The smaller increase in d_u is mainly caused by to the concentration of deformation (damage) in a single storey (usually the first storey) at the ultimate limit state of URM buildings as well as by the reduction in the ultimate drift ratio capacity of walls due to the increase in their pre-compression level; see Eq. 4. Hence, taking the number of stories into account in proposing q_0 values for URM buildings (see Section 4.4) is not only statistically but also theoretically justifiable.

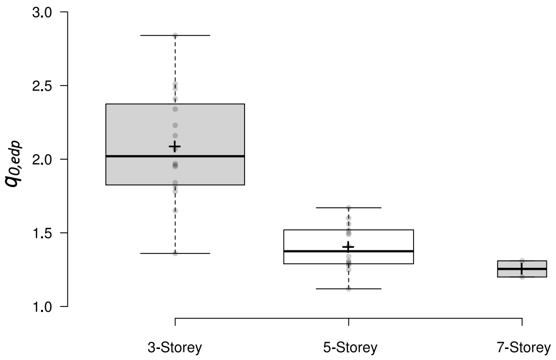

Figure 4: Influence of the number of stories on q_0

Conclusion

The adaptive force-based procedure was introduced as an alternative to current force-based procedures for the seismic design of masonry buildings. Unlike conventional force-based procedures that rely on over conservative behaviour factors prescribed by codes, the adaptive force-based procedure estimates the values of the overstrength ratio, and therefore the behaviour factor, individually for each building using pushover analysis. By eliminating the uncertainties associated with the overstrength ratio, the adaptive force-based procedure can provide designs that are much less conservative than those provided by conventional force-based procedures. As discussed in this paper, the potential of masonry has not yet been exhausted and there is a clear room for future developments. Until the displacement-based approach becomes a standard method in practice, the adaptive force-based procedure can provide the engineering community with a proper tool for a better utilisation of the sustainability of masonry in the construction of new buildings.

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Annex A

The following figures show the storey plans P1 to P6; see also Table 1.

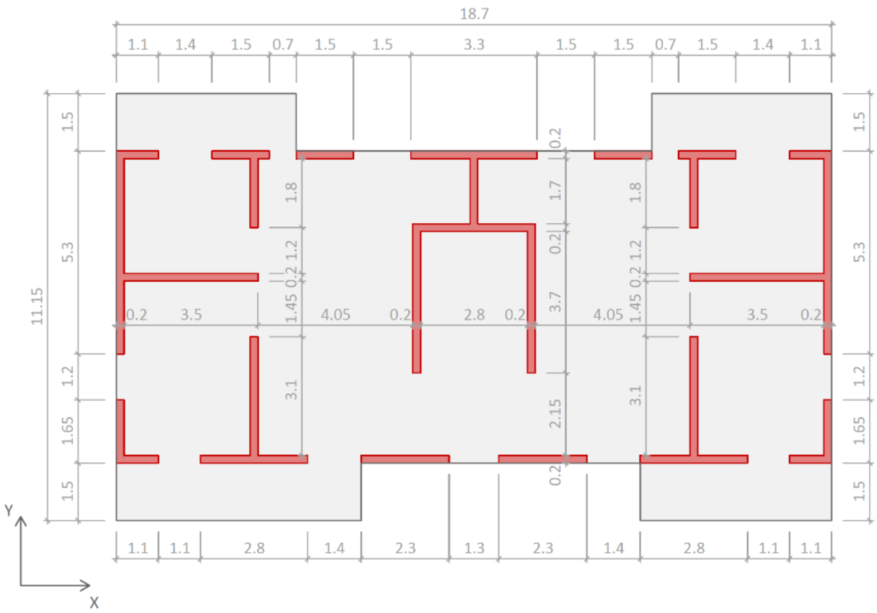


Figure A1: Plan P1 (from [12])

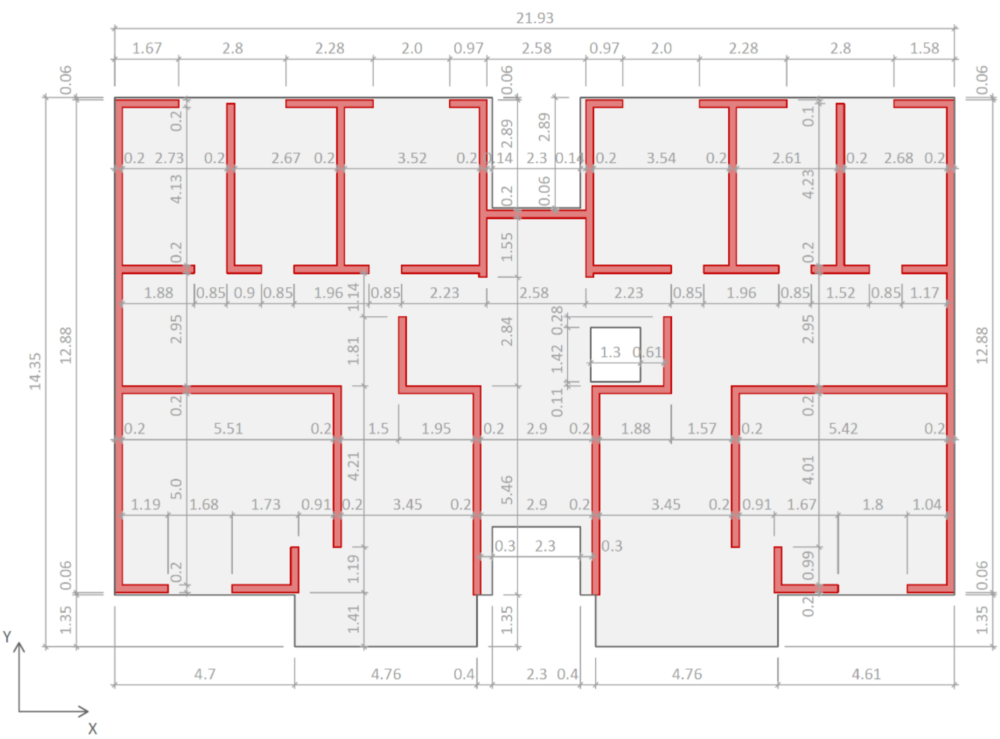


Figure A2: Plan P2 (from [12])

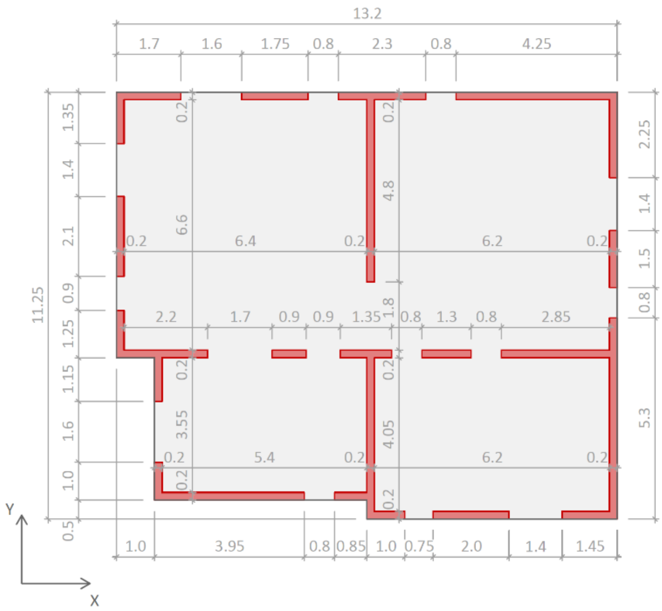


Figure A3: Plan P3

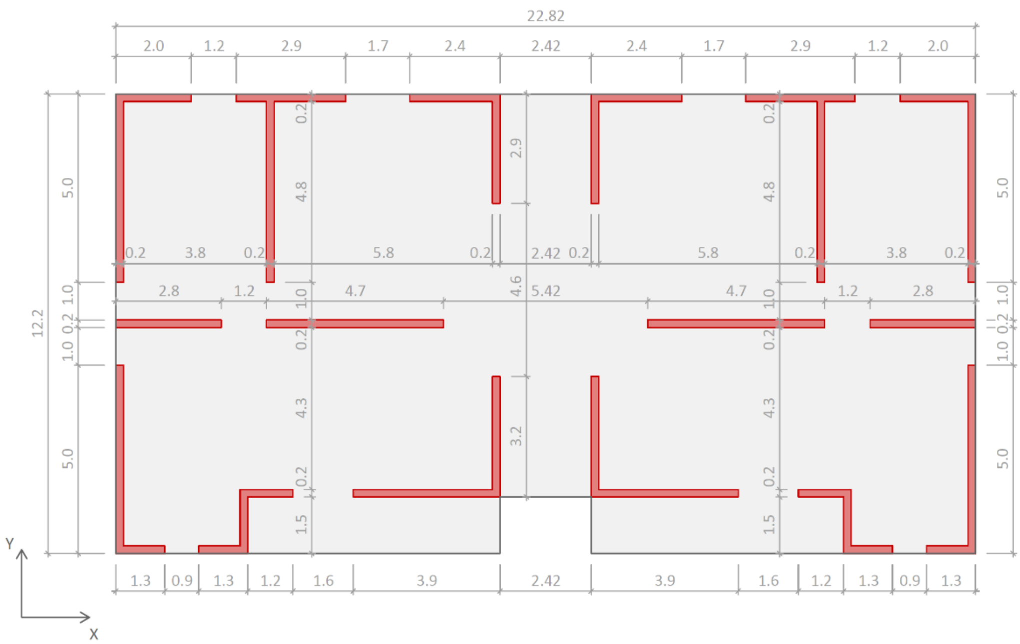


Figure A4: Plan P4 (from [12])

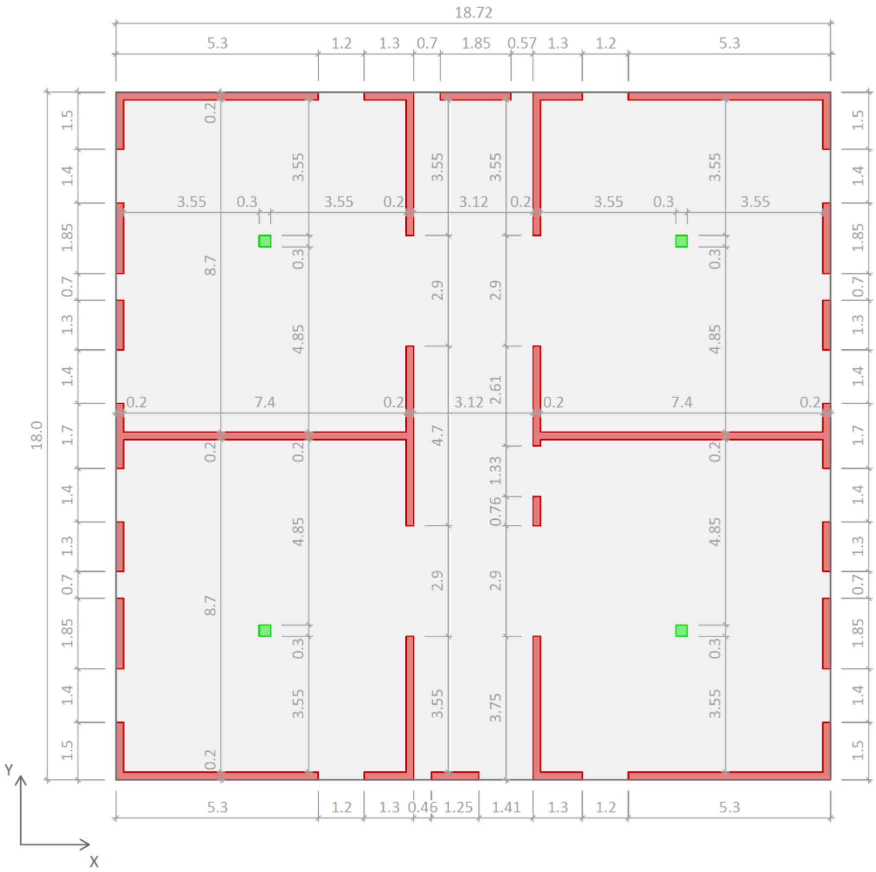


Figure A5: Plan P5; Columns are 0.3m x 0.3m (from [12])

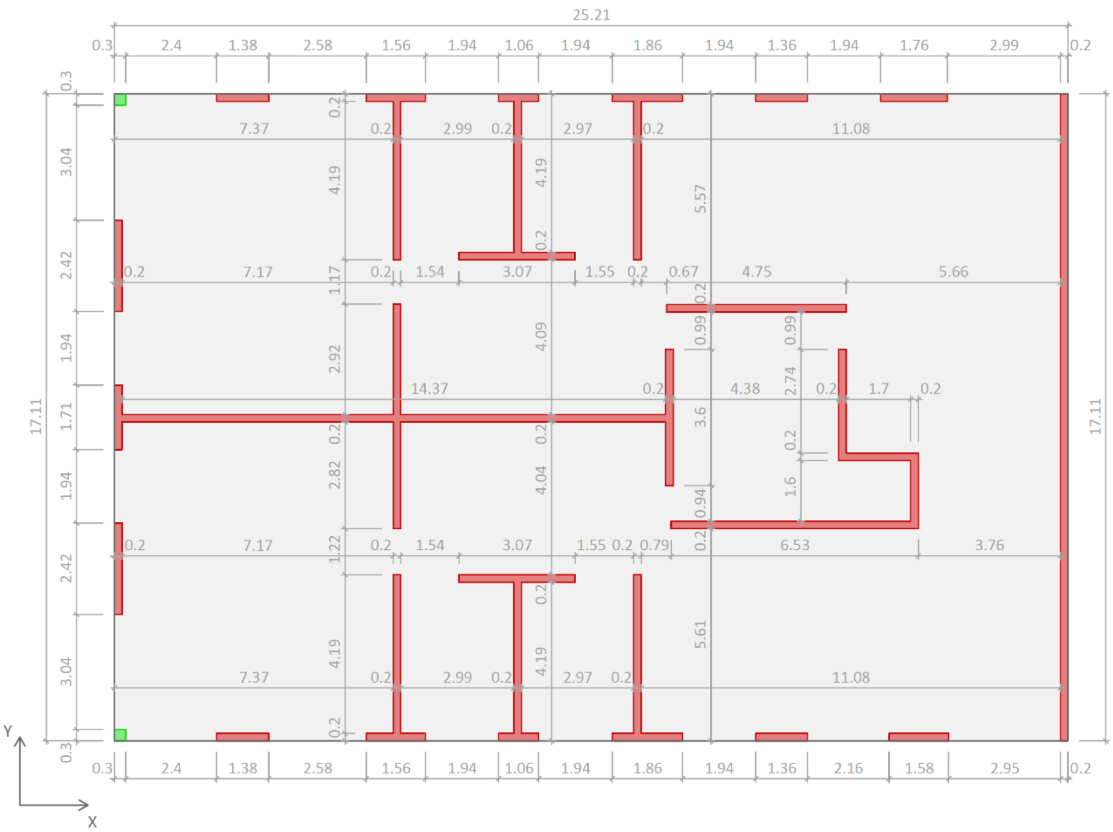
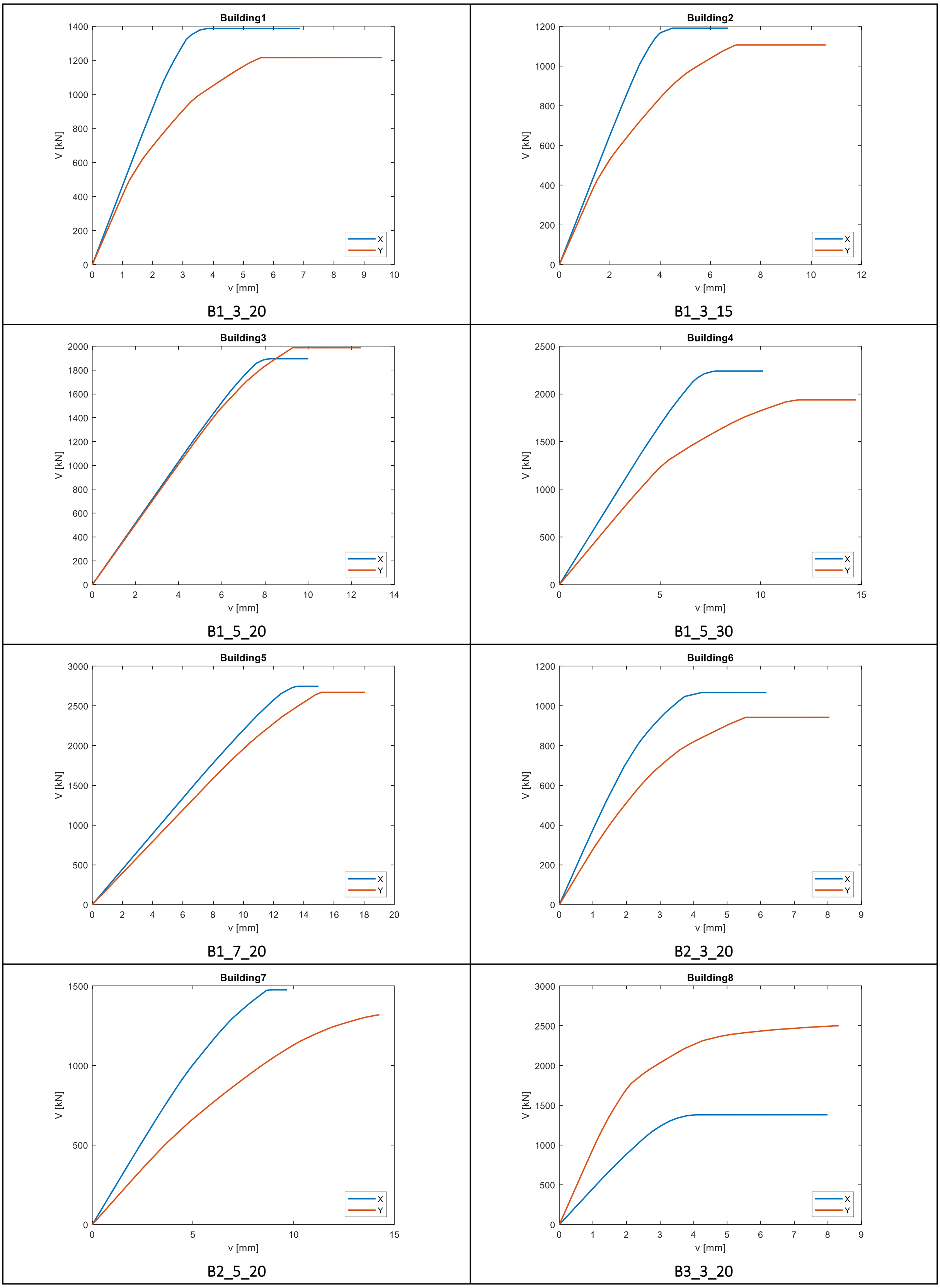
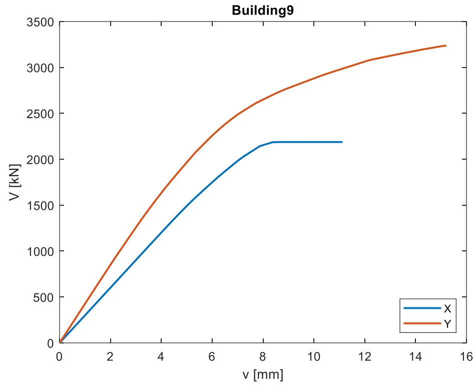


Figure A6: Plan P6; Columns are 0.3m x 0.3m

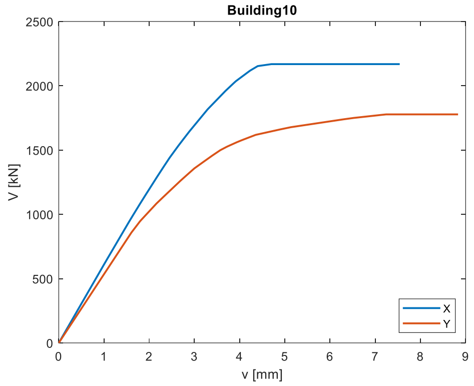
Annex B

The figure below shows the capacity curves of the studied buildings. Note that the shown capacity curves are those corresponding to the $q_{0,edp}$ values reported in Annex C.

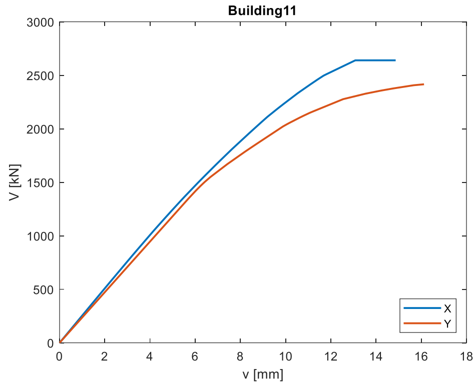




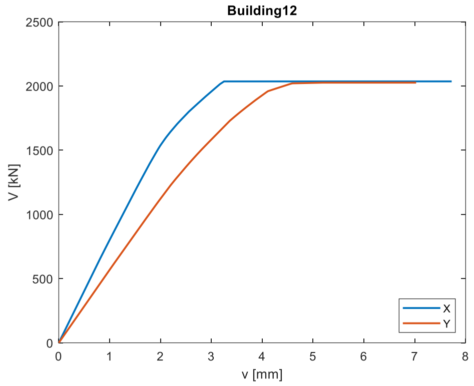
B3_5_20



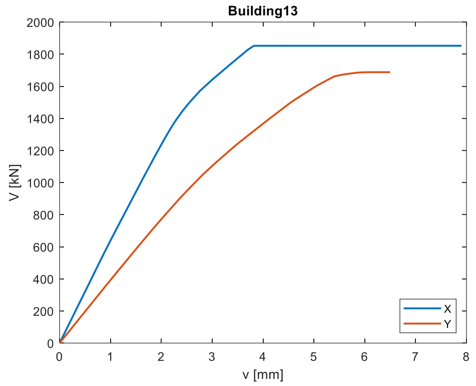
B4_3_20



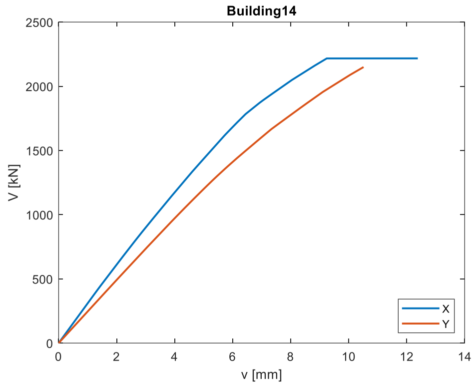
B4_5_20



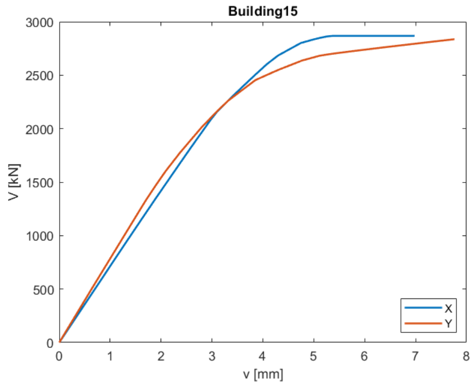
B5_3_20



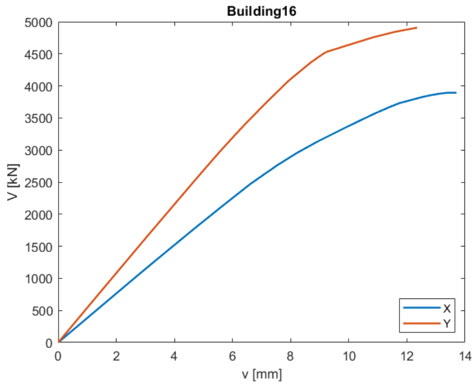
B5_3_15



B5_5_20



B6_3_15



B6_5_20

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Figure B1: Inventory of the building capacity curves

Annex C

The following table presents the parameters of idealised capacity curves as well as the estimated values of q_0 and *OSR* for each building. It should be noted that for each principal direction of the buildings, eight pushover analyses were performed, and accordingly, eight sets of q_0 and *OSR* values were obtained. The values reported here are the minimum of those values and do not necessarily correspond to the same pushover analysis.

Table C1: Parameters of idealised capacity curves and estimated values of q_0 and *OSR*

Building	Dir.	*Fmax* [kN]	*dy* [mm]	*du* [mm]	*T* [s]	$q_{0,edp}$[(1)]$=\mu$	$q_{o,eep}$[(2)]	$q_{0,EC8}$, Ground Type A	B, E	C	D	*OSR*
B1_3_20	X	1387	3.09	6.87	0.23	2.23	1.86	1.69	1.56	1.46	1.35	1.64
	Y	1216	3.99	9.61	0.22	2.41	1.95	1.76	1.61	1.51	1.38	2.32
B1_3_15	X	1190	3.76	6.71	0.26	1.78	1.60	1.51	1.41	1.34	1.25	1.64
	Y	1105	5.11	10.57	0.25	2.07	1.77	1.66	1.53	1.44	1.33	2.11
B1_5_20	X	1895	7.44	10.01	0.39	1.34	1.30	1.34	1.27	1.23	1.17	1.66
	Y	1987	8.17	12.46	0.36	1.52	1.43	1.48	1.39	1.32	1.24	2.19
B1_5_30	X	2240	6.74	10.11	0.36	1.50	1.41	1.45	1.36	1.30	1.23	1.66
	Y	1935	8.79	14.71	0.34	1.67	1.53	1.56	1.45	1.37	1.28	2.56
B1_7_30	X	2746	12.49	14.97	0.45	1.20	1.18	1.20	1.18	1.15	1.11	1.81
	Y	2670	13.83	18.05	0.48	1.31	1.27	1.31	1.30	1.25	1.18	2.32
B2_3_20	X	1067	3.15	6.17	0.24	1.96	1.71	1.57	1.46	1.38	1.29	3.81
	Y	945	4.15	8.07	0.23	1.95	1.70	1.55	1.44	1.37	1.28	3.22
B2_5_20	X	1477	7.54	9.66	0.41	1.28	1.25	1.28	1.23	1.19	1.14	3.96
	Y	1320	10.98	14.25	0.40	1.30	1.26	1.30	1.24	1.20	1.15	4.28
B3_3_20	X	1381	3.22	7.99	0.22	2.48	1.99	1.83	1.66	1.55	1.41	2.02
	Y	2502	3.57	8.33	0.17	2.34	1.92	1.58	1.47	1.39	1.29	2.55
B3_5_20	X	2187	7.49	11.12	0.39	1.49	1.40	1.48	1.38	1.32	1.24	2.04
	Y	3234	9.50	15.19	0.31	1.60	1.48	1.47	1.38	1.31	1.23	3.77
B4_3_20	X	2168	3.82	7.54	0.24	1.97	1.72	1.59	1.47	1.40	1.30	3.52
	Y	1782	4.11	8.85	0.23	2.16	1.82	1.65	1.52	1.43	1.33	1.88
B4_5_20	X	2635	11.29	14.84	0.38	1.31	1.28	1.30	1.24	1.20	1.15	3.57
	Y	2412	11.38	16.10	0.40	1.41	1.35	1.41	1.33	1.28	1.21	2.12
B5_3_20	X	2036	2.72	7.73	0.23	2.84	2.16	2.07	1.85	1.71	1.53	3.08
	Y	2026	3.82	7.03	0.28	1.84	1.64	1.58	1.46	1.39	1.29	3.45
B5_3_15	X	1852	3.15	7.90	0.26	2.51	2.01	1.97	1.77	1.65	1.48	3.28
	Y	1688	4.76	6.50	0.27	1.36	1.31	1.24	1.20	1.16	1.12	3.67
B5_5_20	X	2217	7.95	12.39	0.38	1.56	1.45	1.53	1.42	1.35	1.26	4.75
	Y	2150	9.39	10.51	0.42	1.12	1.11	1.12	1.10	1.08	1.06	4.09
B6_3_15	X	2867	4.24	6.98	0.29	1.65	1.51	1.46	1.37	1.31	1.23	3.06
	Y	2841	4.30	7.77	0.24	1.81	1.62	1.48	1.38	1.32	1.24	5.48
B6_5_20	X	3900	10.97	13.73	0.40	1.25	1.23	1.25	1.20	1.17	1.13	2.16
	Y	4907	9.57	12.35	0.43	1.29	1.26	1.29	1.25	1.21	1.16	5.99
No. of data						32	32	32	32	32	32	32
min						1.12	1.11	1.12	1.10	1.08	1.06	1.64
max						2.84	2.16	2.07	1.85	1.71	1.53	5.99
mean						1.73	1.55	1.50	1.40	1.34	1.25	2.99
stdev						0.45	0.28	0.22	0.17	0.14	0.11	1.15

(1) Equal displacement principle
(2) Equal Energy principle