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NEW SEISMIC DESIGN PROVISIONS IN THE SWISS STRUCTURAL MASONRY CODE

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Abstract

In 2003, the Swiss Society of Engineers and Architects (SIA) introduced a set of new structural design codes including the masonry code SIA 266:2003. After a decade in service, this set of structural codes underwent a revision. A new, revised version of the masonry structural code was published in July 2015. The revision had two main objectives. Firstly, the provisions introduced by SIA 266:2003 had to be updated and made more user-friendly. Secondly, the new code had to be compatible with its companion codes SIA 260 to 267 as well as with Eurocode 6 (EN 1996), Design of Masonry Structures, though considering specific national requirements. Major changes introduced during revision concerned the seismic design of masonry shear walls. The design philosophy has been modified towards the performance-based design. In addition to the established force-based seismic design, deformation-based design has been introduced in the code, though with some limitations regarding the level of the vertical load acting on the walls. Overall, the new code permits a better utilization of the potential offered by structural masonry.

Keywords: displacement capacity; masonry; seismic design; shear wall; structural code.

1. Introduction

The revised Swiss masonry code SIA 266:2015 [1] combines the latest European developments with the Swiss code tradition and provide a good basis for structural masonry design. In line with the Swiss code tradition, the revised SIA 266:2015 is a very concise document. Users are required to have a sufficient theoretical training and practical experience in design and construction to be able to apply the code as responsible professional engineers. Further, the structural codes in Switzerland are not part of the law per se, but represent the state of the art of the corresponding knowledge and methods. The revision of the previous version of the masonry code, SIA 266:2003 [2], had two main objectives. Firstly, the provisions introduced by SIA 266:2003 had to be updated and made more user-friendly. Secondly, the new code had to be compatible with its companion codes SIA 260 to 267 as well as with Eurocode 6 (EN 1996), Design of Masonry Structures, though considering specific national requirements. Major changes introduced during revision concerned the seismic design of masonry shear walls. The design philosophy has been modified towards performance-based design. In addition to the established force-based seismic design, the deformation-based design has been introduced in the code, though with some limitations with respect to the level of the vertical load acting on the walls. It should be noted here that the Swiss structural masonry code covers only the design of new masonry structures. Further, the supplementary specifications (mostly material testing procedures) are given in SIA 266/1:2015 [3]. The assessment of existing masonry structures is governed by separate documents and regulations.

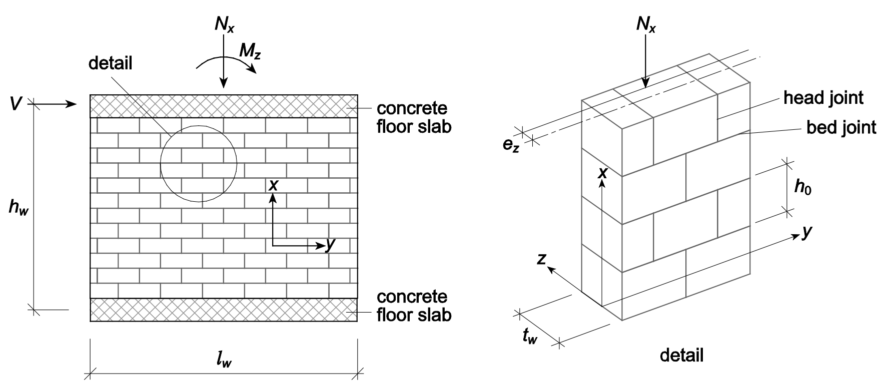

Fig. 1 – Typical masonry wall and notation [1]

The revised code defines so-called standard masonry, which must satisfy certain (minimal) requirements with regard to masonry material characteristics (particularly masonry strength in the *x* direction, see also Fig. 1 for notation). If these requirements are not met (i.e. with lower or higher values), the material characteristics of such masonry must be declared. In the revised code, the thin layer mortar masonry is now classified as standard masonry with the condition that flexure failure (bending about the *y* axis) of such masonry takes place in the joints and not through the masonry units. Further, the revised code introduces the possibility to design masonry elements subjected to axial and/or shear loading using simple and enhanced verifications. The latter can be performed using design charts or a general procedure. This paper, however, concentrates on the above-mentioned seismic design provisions, which will be presented in detail in the following.

2. Seismic design provisions

Seismic design provisions in the previous version of the code were not very comprehensive and included the thickness and slenderness limits only [2]. The minimal thickness of the load-carrying shear wall, t_w, was limited to 150 mm and its slenderness ratio, h_w/t_w, to 17, cf. Fig. 1 for notation. Further, the behavior factor, q, was set to 1.5 for unreinforced and 2.5 for reinforced or, as it was called in the code, ductile masonry. Fig. 2 shows a sketch of the orthogonally laid reinforcement requested for so-called ductile masonry. The sum of geometrical (steel) reinforcement ratios in both x and y directions shall be at least 0.2%, whereas each of them shall not be smaller than 0.05%. In the edge regions (width of edge zone at least 250 mm) the geometrical reinforcement ratio, ρ, shall exceed 0.3%, see also Fig. 2. All reinforcement must be properly anchored (in adjacent structural members where appropriate) corresponding to the flow of the forces.

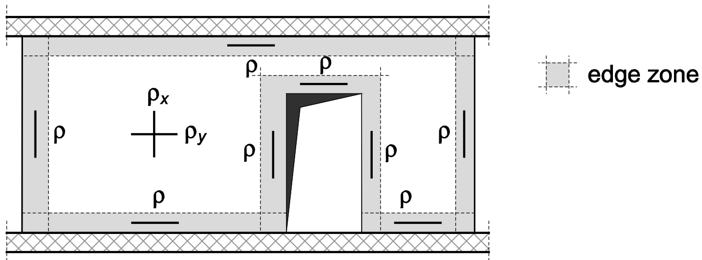

Fig. 2 – Ductile masonry reinforcement according to SIA 266:2003 [2]

In the revised version of the code the concept of ductile masonry has been abandoned. The new version introduces so-called masonry with increased deformation capacity, which has to fulfill certain performance goals and need not necessarily to be reinforced with (classical) steel reinforcement, see section 2.4 for a description.

Cross sectional forces and deformations shall be determined on the basis of linear elastic theory using average stiffness values. The non-linear behavior is taken into account by the previously mentioned behavior factor. The code requires that masonry walls are designed for both in- and out-of-plane seismic actions.

2.1 Stiffness of masonry walls

One of the most challenging and still unsatisfactorily solved issues in the seismic design of structural masonry is the assessment of the stiffness of shear walls. Determining the correct stiffness is of utmost importance for the deformation-based design of masonry structures. In general, the response of masonry walls subjected to cyclic shear is nonlinear and depends on several parameters, e.g. the level of the pre-compression applied to the shear wall, boundary conditions, wall slenderness, etc. Moreover, the reduction in both strength and stiffness of masonry can be observed during cyclic loading, see e.g. Mojsilović [4]. Usually, the hysteresis envelope (shear force-horizontal displacement relationship) is chosen as the representative load-deformation characteristic for the evaluation of the deformation capacity of masonry (Fig. 3). This relationship can be modelled by a (bilinear) linear-elastic ideal-plastic curve. The first portion of this curve is determined by the effective stiffness, K_{eff}, see Fig. 3.

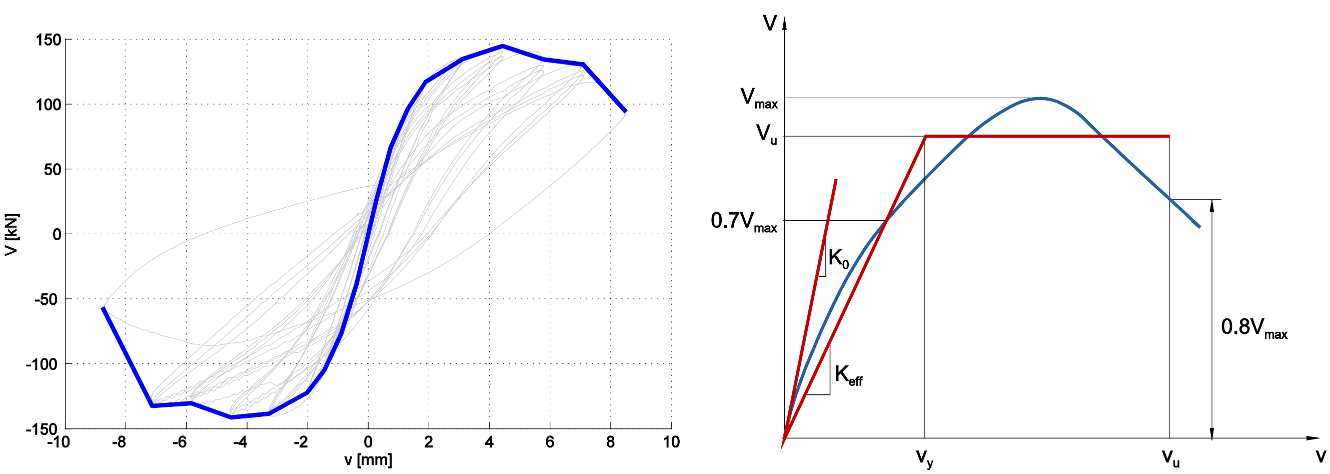

Fig. 3 – Typical hysteresis envelope and its bilinear idealization

In general, the effective stiffness, is a complex parameter and difficult to determine. For practical applications, it is usually adopted as a certain percentage (usually 50%) of the elastic stiffness, K_{el}. Elastic stiffness is usually calculated on the basis of elastic beam theory incorporating shear deformation. Furthermore, the masonry material's mechanical characteristics involved in this calculation are rarely determined through material tests, but instead are usually based on experience or taken from structural code provisions and recommendations.

If experimental data from cyclic tests is available, the tangent stiffness, K_0, evaluated as the slope of the line connecting the positive and negative extreme points of the first hysteresis loop, i.e. the loop corresponding to the first applied displacement cycle, see also Salmanpour et al. [5], could be used to obtain the effective stiffness. However, as shown in Fig. 4, the ratio between the tangent stiffness, K_0, and the effective stiffness as well as between the elastic stiffness and the effective stiffness for clay block masonry walls, exhibits scattering and the latter ratio is, in most cases, much smaller than 0.5. Furthermore, as can be clearly seen from the hysteretic behavior of the masonry walls a significant degradation of stiffness K_0 during cyclic loading can be observed. Fig. 4 shows exemplarily the stiffness degradation for three tests of the Series T from Salmanpour et al. [5].

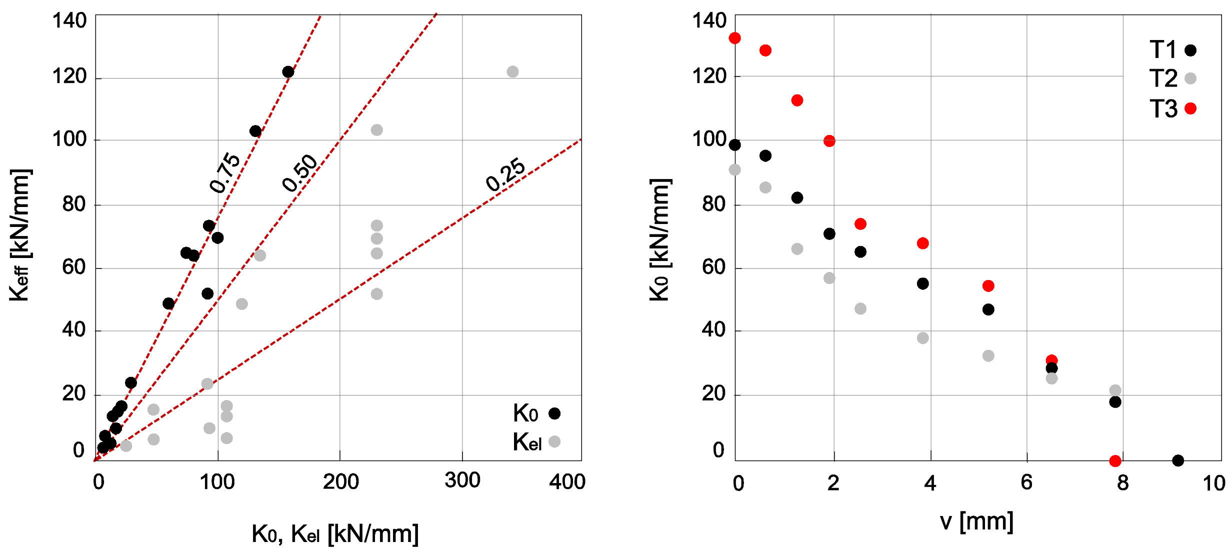

Fig. 4 – Stiffness comparisons and degradation [4]

Swiss structural masonry code SIA 266:2015 requires, if there is no experimental data on stiffness available, to choose the effective stiffness when using the force-based method for design between 40% and 60% of the value of the elastic stiffness. When using the deformation-based method, the range between 20% and 40% of K_{el} is suggested for the effective stiffness. In such a way it is ensured that the results are rather on the conservative side (i.e. on the safe side), since an overestimation of the stiffness in the case of the force-based method results in larger equivalent forces and an underestimation of the stiffness in the case of the deformation-based method results in larger displacements.

2.2 Behavior factor (*q*-factor)

The ability of a structure to resist seismic action in the non-linear range with over-strength may be considered in design by a reduction of the elastic response spectrum with the behavior factor, *q*. The behavior factor depends on the plastic deformation and energy dissipation capacity of the structure and its elements. The use of linear methods of analysis and (equivalent) force-based design with the behavior factor is considered by the masonry research community to be too conservative and to cause some inconsistencies in the design of masonry buildings. The main issue is that the capacity of a structural element, e.g. a wall, can be reached for seismic loading that is much lower than that corresponding to the capacity of the whole building.

According to the revised code SIA 266:2015 the behavior factor shall be taken, in general, as 1.5. If the building layout satisfies additional criteria of regularity as defined by SIA 261:2014 [6] (uniformity and symmetry in regard to horizontal stiffness and mass distribution; in-plane stiffness of the floor slabs much higher than that of the walls; limited area of opening and recesses; structural members resisting horizontal forces run without interruption from the foundation to the top of the building) and if the relationship

$$\frac{N_{xd}}{l_w t_w f_{xd}} \leq 0.20 \tag{1}$$

is satisfied, the behavior factor can be set to 2.0. In Eq. (1) l_w and t_w are, respectively, length and thickness of the masonry wall. f_{xd} is the design value of the masonry compressive strength perpendicular to the bed joints and N_{xd} is the design value of the vertical force acting on the wall. This limitation of the vertical load acting on the shear wall ensures that the masonry wall would not exhibit unwanted brittle behavior. The vertical force limit in Eq. (1) has been estimated from the evaluated experimental data. Finally, for masonry with increased deformation capacity the behavior factor of 2.5 may be used.

2.3 Deformation-based design

Unlike its previous version, the revised version of the code allows the use of deformation-based methods for the design of structural masonry. Additional information on deformation-based structural analysis and design is given in the (informative) Annex B of the revised code. In general, it is noted that there are several possibilities (methods) of performing deformation-based analysis and design and that one possible method that can be used is given in EN-1998-1:2004 [7]. In particular, the Annex provides some recommendations regarding deformation-based analysis and the design of unreinforced masonry (URM) structures:

(i) For the masonry material properties linear-elastic ideal-plastic behavior may be assumed and the stiffness shall be assumed as described earlier (see sub-chapter 2.1). The ultimate shear strength shall be determined according to the SIA 266:2015 provisions (application of the overlapping discontinuous stress fields).

(ii) For shear walls a simple translation failure mechanism may be assumed, whereby the horizontal deformations are limited according to the code provisions, see sub-chapter 2.3.1.

(iii) For the design of the walls for in-plane loading, the eccentricity of the vertical load (e_z in Fig. 1) and possible floor slab rotations have to be taken into account. The out-of plane design of single walls may be performed independently.

(iv) The analysis shall be performed using characteristic values of the material properties. At the same time, the deformation capacity of the structural element shall be reduced by a partial safety factor γ_D = 2.0.

(v) The failure of the structural element takes place when either the ultimate resistance or the ultimate horizontal displacement is reached.

2.3.1 Displacement capacity

The (ultimate) displacement capacity is actually the displacement corresponding to the limit state of Near Collapse (NC) in accordance with EN 1998-3:2005 [8]. In the NC limit state, a URM wall is severely damaged, with low residual lateral strength and stiffness. However, the wall is still capable of sustaining vertical loads. Researchers have used various approaches to estimate the displacement capacity of different structural elements and there is no consensus within the research community on an approach for estimating the displacement capacity. However, the widely accepted approach for most of structural members including URM walls is to estimate the displacement capacity as the displacement corresponding to 20% strength degradation, v_u, see Fig.

3. The story drift ratio (horizontal displacement divided by the wall height) capacity of walls that fail in shear is particularly small, e.g. the maximum and minimum values obtained in a recent experimental study [5] were 0.32% and 0.23%, respectively. However, the walls that failed in flexure exhibit greater drift capacity [5].

Regarding the above-mentioned procedure used for the estimation of the displacement capacity, it should be mentioned that although it has been widely used by the majority of researchers and has been adopted by most of the current codes and guidelines, it was not found to be completely consistent with the behavior of contemporary unreinforced masonry walls [5]. In fact, the procedure does not provide a uniform margin of safety against collapse for all ranges of URM walls. The procedure relies on the assumption that most of the structural elements have some capacity for deformation beyond the peak of the strength-deformation relation with a reduction in strength. However, as shown by the test results [5], some of the walls that failed in shear, exhibited limited strength degradation after the peak strength and before collapse. In such cases, the procedure did not fully correspond to the NC limit state of the walls, because the displacement corresponding to 20% strength degradation coincided with the collapse of the walls. Hence, the procedure did not provide any margin of safety against the collapse and overestimated the displacement capacity of the walls. By contrast, some other walls that failed in flexure and in shear, respectively, had a considerable residual capacity for further increase of displacement after 20% strength degradation.

Therefore, in order to take advantage of the complete displacement capacity of URM walls and to avoid an unsafe design, it would be necessary to develop a more consistent procedure for estimating the displacement capacity of URM walls. Such a procedure should directly refer to the true ultimate limit state of URM walls, i.e. the inability to carry applied vertical loads. For instance, a uniform margin of safety against collapse can be provided by applying a safety factor for the displacement corresponding to collapse. However, it should be noted that the available experimental data on the ultimate limit state of URM walls is very limited, because the limit state that refers to the inability to carry imposed vertical loads has generally been avoided due to safety issues. An alternative approach for unreinforced masonry walls could be to estimate the displacement capacity based on the vertical stiffness degradation.

Taking into account the above-mentioned observations, the revised code SIA 266:2015 recommends, when the above-mentioned experimental data is not available to use particular limits for the design values of the ultimate story drift ratios (ultimate horizontal displacement divided by the wall height, i.e. v_u/h_w). If the shear wall spans, i.e. is fixed, between stiff reinforced concrete floor slabs the recommended design value of the ultimate story drift, δ_{ud}, equals to 0.2%. In other cases (e.g. with a cantilever or partial fixation boundary condition) the value of 0.4% is recommended. When applying these recommended values, the requirement given by Eq. (1) must also be satisfied.

2.4 Masonry with increased deformation capacity

The revised code foresees the use of masonry with increased deformation capacity for construction work class III (structures with vital infrastructure functions and those with considerable environmental risk) in all seismic zones and for the construction work class II (structures where a large public gathering is possible, structures with important infrastructure functions and those with limited environmental risk) in the highest seismic zone. The seismic zones in Switzerland are defined in SIA 261:2014 [6].

Unlike in the previous version of the code, masonry with increased deformation capacity need not necessarily be reinforced with steel reinforcement. Other metallic and non-metallic reinforcement systems may be applied. Even unreinforced masonry, e.g. with special construction detailing, could be applied. The revised code sets the performance goal, according to which the ultimate story drift ratio of the shear wall, δ_u, larger than 2.0% must be reached (regardless of whether the masonry is reinforced or not). In order to verify this, standard static-cyclic tests must be performed. In addition, the hysteretic envelope must exhibit a non-linear behavior, i.e. after the elastic portion of the curve, a plateau and/or softening branch must be exhibited, see also Fig. 3.

The test protocol must include the shear wall boundary conditions, the level of the vertical load applied, the wall slenderness, h_w/l_w, and the shear load-horizontal displacement curve (hysteresis). If the shear wall is reinforced, the reinforcement ratio as well as the arrangement and anchorage of the reinforcement must be provided.

3. Code implementation

After the release of the revised code, a series of introductory courses were given. In addition, a special code committee working group drafted the background document [9] that should ease the use of the revised code. This document explains selected code provisions in detail, gives the theoretical background behind the different clauses and presents several examples. These examples concentrate on the code design methods and in particular on the application of new seismic provisions. The following example, which has been adapted from [9], presents the seismic design of a typical Swiss family house.

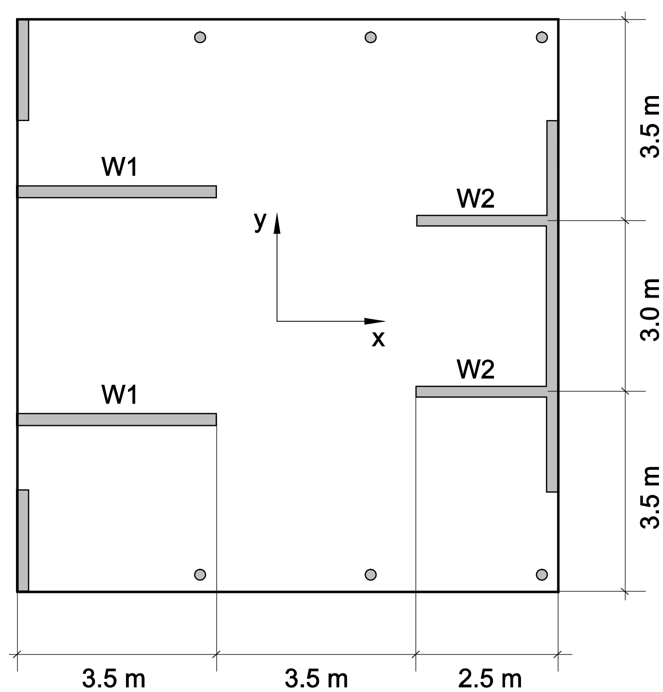

Fig. 5 – Floor plan

In this section, the force-based and deformation-based design methods are compared for an exemplary two-story residential building located in Basel, Switzerland. The building consists of clay block URM walls and 200 mm thick reinforced concrete slabs. Fig. 5 shows the floor plan of the building. The walls' height, h_w, equals 3.0 m and their thickness, t_w, 200 mm. The design values of masonry compressive strengths perpendicular, f_{xd}, and parallel, f_{yd}, to the bed joints are 3.5 MPa and 1.6 MPa, respectively. Characteristic values of the elasticity modulus, E_{xk}, and shear modulus, G_k, are 7.0 GPa and 2.8 GPa, respectively. The design value of the friction coefficient in the bed joint, μ_d, is 0.6. The actions on the structure are summarized in Tables 1 and 2.

Table 1 – Vertical loads acting on walls W1 and W2 (in kN)

Story	**Height [m]**	***m* [kg]**	**Q_d [kN]**	**W1**	**W2**
2	6.00	81142	796	126	86
1	3.00	79001	775	124	84
Total		160143	1571	250	170

Table 2 – Parameters of the elastic response and design spectrum, according to [6] and [1]

Seismic zone	Zone	[-]	Z3a
	a_{gd}	[m/s^2]	1.30
Construction work class	BWK	[-]	I
Ground	Class	[-]	B
	S	[-]	1.20
	T_B	[s]	0.15
	T_C	[s]	0.50
	T_D	[s]	2.00
Elastic response spectrum	ξ	[-]	0.05
	η	[-]	1.00
Design spectrum	γ_f	[-]	1.00
	g	[m/s^2]	9.81
	q	[-]	2.00

In Table 2, the parameters of the elastic spectrum, S, T_B, T_C and T_D, are given, and ξ and η are the viscous damping factor and its correction factor, respectively. The importance factor is denoted by γ_f. The design value of the ground acceleration corresponding to the chosen seismic zone is denoted by a_{gd}.

3.1 Structural analysis

The torsional effects are neglected and the analysis is carried out separately for each principal direction, i.e. x and y (see Fig. 5), using two-dimensional frame models. However, only the results for the x direction are presented here. As required by SIA 266:2015 (see sub-chapter 2.1), the effective stiffness of the walls are assumed to be 50% and 30% of their elastic stiffness values for the force-based and deformation-based methods, respectively. Regarding the slabs, an effective width of $3t_w$ is assumed as suggested by Priestley et al. [10]. Note that the pre-compression level, as defined by the left hand side of Eq. (1) equals to 0.10 for the lower story walls.

3.2 Force-based design

3.2.1 Seismic demand

The equivalent force method is used for the force-based design of the building according to SIA 261:2014 [6]. The fundamental period of vibration of the building is estimated as $T_1 = 0.050\ h^{0.75} = 0.05 \cdot 6^{0.75} = 0.19$ s, where h is the height of the building (in m). Thereafter, the design base shear, F_d, and its distribution over the height of the building, F_{di}, are determined as follows:

$$T_B \leq T_1 = 0.19 \leq T_C \rightarrow S_d\left(T_1\right) = 2.5 \frac{a_{gd}}{g} \frac{S}{q} = 0.2 \tag{2}$$

$$F_d = S_d\left(T_1\right) \sum_j \left(Q_d\right)_j = 312 \text{ kN} \tag{3}$$

$$F_{di} = \frac{z_i (Q_d)_i}{\sum_j z_j (Q_d)_j} F_d \rightarrow F_{d1} = 102 \text{ kN}, F_{d2} = 210 \text{ kN} \tag{4}$$

In the above equations, z_i denotes the height of the ith storey from the base level and $S_d(T_1)$ represents the ordinate value of the design spectrum at the fundamental period of the building. Note that the behaviour factor, q, was set to 2.0 since the structure satisfies the corresponding requirements of SIA 266:2015, see sub-chapter 2.2. It is noteworthy that the same value for the fundamental period of the structure, i.e. 0.19 s, is obtained from the modal analysis of the structure.

Table 3 – Design value of the forces acting in the plane on walls in the first story

Wall	N_{xd} **[kN]**	h_s* **[m]**	V_d **[kN]**	M_{z1d} **[kNm]**	M_{z2d} **[kNm]**
W1	231	4.45	102	148	454
W2	189	3.76	54	41	203

*h_s is the shear span, i.e. the height of the inflection point, and equals M_{z2d}/V_d.

Table 3 shows the design values of the forces acting in the plane of walls W1 and W2 in the first story. These values were obtained from the structural analysis as described in sub-chapter 3.1. See Fig. 6 for the notation of the actions.

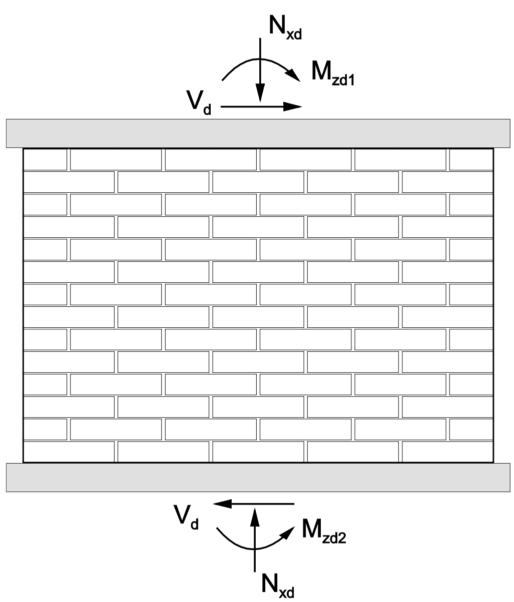

Fig. 6 – In-plane forces acting on the wall

3.2.1 Seismic capacity

Table 4 presents the shear strength, V_{Rd}, of the walls determined using Eq. (5) according to SIA 266:2015.

$$V_{Rd} = \frac{f_{yd} l_w t_w N_{xd} \mu_d}{N_{xd} + N_{xd}\mu_d^2 + 2 f_{yd} t_w h_s \mu_d}, \quad \mu = \tan\alpha = \frac{2V_{Rd} h_s}{N_{xd} h_w} > \mu_d \tag{5}$$

Table 4 – Shear strength of the walls

Wall		W1	W2
V_{Rd}	[kN]	77	53
$tan\alpha$	[-]	1.00	0.70
μ_d	[-]	0.60	0.60
$tan\alpha > \mu_d$	[-]	OK	OK
V_{Rd} / V_d	[-]	0.75	0.98

It can be seen from Table 4 that the building does not satisfy the strength requirement according to the implemented force-based design method. In fact, the design would be acceptable only with a behavior factor, q, greater than 2.7. Furthermore, it is noteworthy that if the frame action is ignored (strong pier-weak spandrel assumption), the building needs a behavior factor of at least 3.0 to satisfy the strength requirement. This is because ignoring the frame action increases the shear span, h_s, and consequently decreases the shear strength, V_{Rd}, of the walls, cf. Eq. (5).

3.3 Displacement-based design

3.3.1 Seismic capacity

Table 5 summarizes the derivation of the capacity curves for walls W1 and W2 as well as for the building, see Fig. 7. The fundamental period, T_1, and mode shape $Ø_1=\{ø_2, ø_1\}^T$ of the structure (in the x direction) were determined as 0.24 s and $\{1.0, 0.4\}^T$.

Table 5 – Parameters of the capacity curves

Wall			W1	W2	Total
Shear span	h_s	[m]	4.13	3.44	
Shear strength	V_{Rd}	[kN]	87	53	2 ($V_{Rd,1}$ + $V_{Rd,2}$) = 280
Yield displacement (first storey)	v^*_y	[mm]	1.35	1.51	
Yield displacement (top storey)	v_y	[mm]	3.38	3.77	
Effective stiffness	K_{eff}	[kN/m]	25792	13962	2 ($K_{eff,1}$ + $K_{eff,2}$) = 79509
Storey drift ratio capacity [1]	δ_{ud}	[%]	0.40	0.40	
Displacement capacity of the building	v_{Rd}	[mm]	14.03	14.26	min ($v_{Rd,1}$, $v_{Rd,2}$) = 14.03

Note that the storey drift ratio capacity, δ_{ud}, was set to 0.4%, since as can be seen from the h_s values in Table 5, which are obtained from the structural analysis, the slabs are not stiff enough to impose a fixed-ends boundary condition, see sub-chapter 2.3. Further, the displacement and stiffness values in Table 5 are estimated using the following equations:

$$v_y^* = \frac{6M_{z1d}h_w^2 + 4V_{Rd}h_w^3}{E_{x,eff}l_w^3 t_w} + \frac{6}{5}\frac{V_{Rd}h_w}{G_{eff}l_w t_w}, \quad E_{x,eff} = 0.3E_{xk} \wedge G_{eff} = 0.3G_k \tag{6}$$

$$v_y = \frac{v_y^*}{\phi_1} \tag{7}$$

$$K_{eff} = \frac{V_{Rd}}{v_y} \tag{8}$$

$$v_{Rd} = \delta_{ud} h_w \left(v_y - v_y^* \right) \tag{9}$$

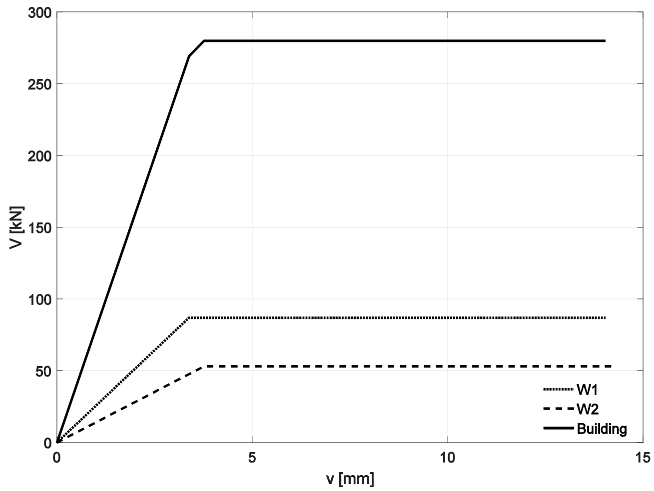

Fig. 7 – Capacity curves

3.3.2 Seismic demand

The structure is approximated by the equivalent SDOF system (subscript $_E$) with following characteristics:

$$T_E = T_1 = 0.24 \text{ s}, \ m_E = \sum_i m_i \phi_i = 112742 \text{ kg}, \ K_E = m_E \left(\frac{2\pi}{T_E} \right)^2 = K_{eff} = 79509 \text{ kN/m} \tag{10}$$

$$\Gamma = \frac{\sum_i m_i \phi_i}{\sum_i m_i \phi_i^2} = 1.2 \tag{11}$$

Afterwards, the displacement demand of the structure, v_d, is estimated as follows, where $S_e(T_1)$ represents the ordinate value of the elastic response spectrum at the fundamental period of the building and v_e is the elastic displacement demand of the building (at the top story).

$$T_B \leq T_1 = 0.24 \leq T_C \rightarrow S_e\left(T_1\right) = 2.5 a_{gd} S \eta = 3.90 \text{ m/s}^2 \tag{12}$$

$$v_e = \Gamma \gamma_f S_e\left(T_1\right) \left(\frac{T_1}{2\pi} \right)^2 = 6.65 \text{ mm} \tag{13}$$

$$T_1 < T_C \rightarrow q_u = \frac{\Gamma \gamma_f S_e(T_1) m_E}{V_{Rd}} = 1.89 > 1 \rightarrow \text{non-linear response} \tag{14}$$

$$v_d = \frac{1}{q_u}\left[1 + (q_u - 1)\frac{T_C}{T_1}\right] v_e = 10.13 \text{ mm} \tag{15}$$

$$\frac{v_{Rd}}{v_d} = \frac{14.03}{10.13} = 1.38 \rightarrow \text{ OK} \tag{16}$$

As can be seen from Eq. (16), the seismic capacity of the structure is 38% larger than its seismic demand according to the deformation-based design method, while the same structure could not be verified using the force-based design method. It is interesting to note that the structure satisfies the deformation-based design requirements even if the frame action is neglected.

4. Conclusions

The revised Swiss structural masonry code SIA 266:2015 provides the structural engineering community with the necessary tools to apply the code as responsible professional engineers. Furthermore, by introducing new and updated provisions it enables a better utilization of the potential offered by structural masonry.

5. Acknowledgement

The help of PhD student, Mr. Amir H. Salmanpour who calculated the example is highly appreciated.

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