

## STATE OF THE SEISMIC DESIGN AND DETAILING OF BUILDINGS WITH RC STRUCTURAL WALLS IN EUROPE

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### SUMMARY

Multistorey buildings, comprising structural systems with flat slabs, small columns designed for gravity loads only and a few relatively slender reinforced concrete structural walls, are popular in Switzerland and central Europe. Such structures can easily be designed for a ductile behaviour at the ultimate limit state and for negligible damage at the serviceability limit state. At the Swiss Federal institute of Technology (ETH) a comprehensive research program was started in 1995 consisting of experimental, theoretical and numerical investigations. The objective of the project is to adapt proven design rules for central Europe with moderate seismicity pertaining to structural walls found in countries with high seismicity. In this contribution results of tests, a simplified analytical model to assess the nonlinear behaviour of structural wall systems and numerical calculations checked against test results and validating the analytical model are briefly presented.

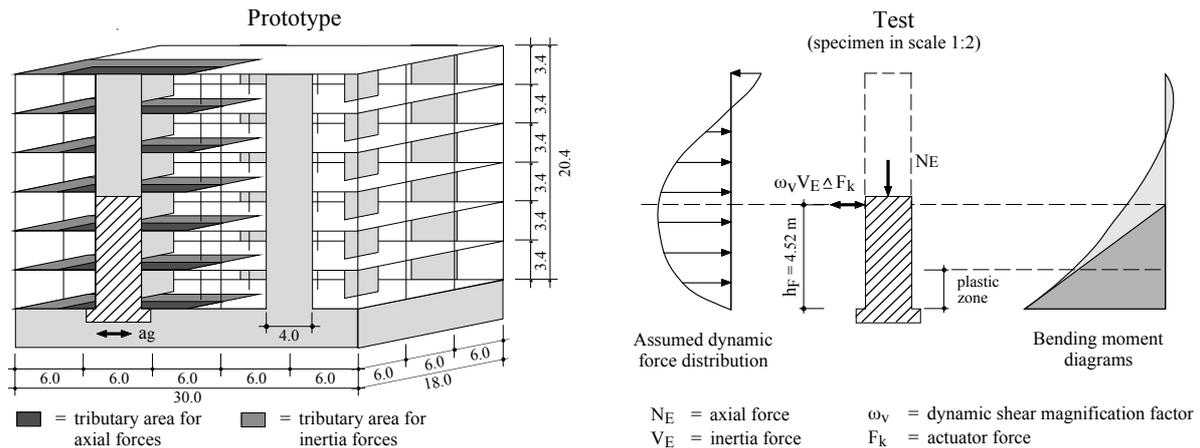
An extensive summary of all test results and all test data are available on the web site of the Institute of Structural Engineering at the ETH (<http://www.ibk.baum.ethz.ch>).

### INTRODUCTION

In Switzerland and in other regions of central Europe with moderate seismicity the earthquake hazard has been underestimated for a long time, but over the last few years it was recognised that much more effort is necessary to make structures safer for seismic actions. An essential need for action exists especially in the case of multistorey office and factory buildings, where life and property of many people can be endangered. These buildings, as shown on the left in Figure 1, are often conceived as structural wall systems consisting of flat slabs, columns designed for gravity loads only and RC structural walls. Flat slabs are beamless concrete slabs typically with spans of 6 to 9 m and thickness of 20 to 30 cm, which are often strengthened around columns to prevent punching shear failure. Columns are in most cases monolithic with the slabs and have a small cross section with typical dimensions ranging from 20 to 40 cm designed to carry axial forces. Structural walls are relatively slender reinforced concrete walls fixed in a very stiff RC foundation box structure with one or more basement storeys. The structural walls have to resist horizontal wind and earthquake forces. Some features concerning conceptual design and advantages of such structural wall systems are summarised in [Bachmann & Dazio, 1997]. Currently, structural wall systems in most cases are firstly planned considering only gravity loads and wind forces and only at a much later stage of the design process is the seismic action taken into account by checking if the structural walls are able to support an equivalent lateral force induced by the earthquake. Neither displacements are checked nor are ductility-improving techniques used. Actually, a drastic improvement in the seismic behaviour of structural wall systems can be easily achieved by means of the capacity design method. In the case of moderate seismicity the structural walls should be designed assuming at most medium ductility, because the walls still have suitable dimensions and the drift limitations imposed by design codes can be easily fulfilled ensuring a better serviceability of the building. The walls are much stiffer than the other structural elements (columns and slabs) so that the latter never yield. For this reason it is sufficient to apply capacity design rules to the walls. As shown in [NZS 3101, 1995], this procedure is absolutely feasible without or with only slight additional costs.

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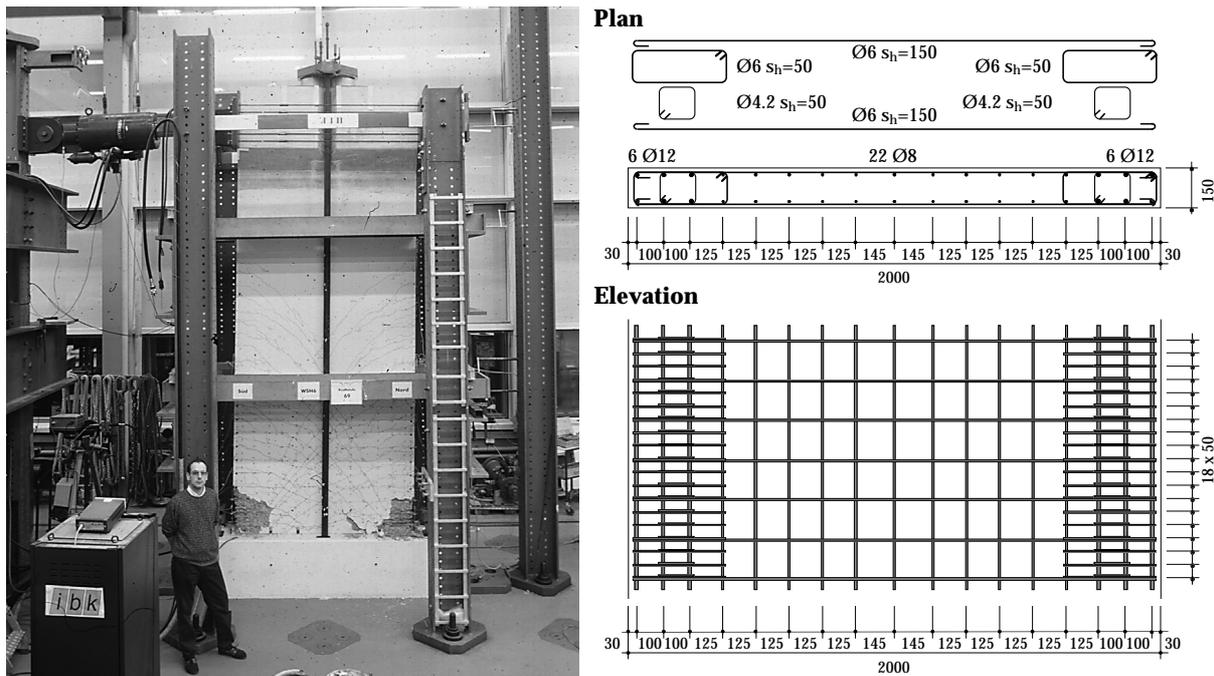


**Figure 1: six-storey reference building (left) and related test set-up (right)**

Moreover, RC members designed according to capacity designed rules can in case of moderate seismicity be provided with a high deformation capacity, actually not needed to fulfil code requirements, in order to mitigate the effects of the great uncertainties that are involved in the description of the seismic input and of strength and deformations capacities of elements and structures. Furthermore, as the plastic mechanism enforced by the designer is single and invariant, simplified nonlinear approaches to assess the seismic behaviour of the structural wall system with suitable reliability can be used. These approaches provide a good control of important parameters like displacement and storey-drift, when the structure enters the nonlinear plastic domain. In Switzerland and in other regions of central Europe the conceptual design and the construction methodologies, the ratio between seismic inertia forces and gravity loads and especially the mechanical properties of the reinforcing steel are quite different as in countries like New Zealand, USA and Japan. For this reason research results gained in these countries with high seismicity cannot be simply applied to central Europe but they have to be adapted. To reach this goal and to give the practising structural designer recommendations to design better structural wall systems a comprehensive research program was started in 1995 at the Swiss Federal Institute of Technology (ETH). It consists of cyclic-static tests on 6 six-storey walls in scale 1:2 [Dazio et al., 1999], shake table tests on 6 three-storey walls in scale 1:3 [Lestuzzi et al., 1999], pseudodynamic tests on 3 three-storey walls in scale 1:3 [Thiele et al., 1999], nonlinear dynamic analyses of structural wall buildings by self-developed computer codes ([Linde, 1993], [Wenk, 1999], [Dazio, 2000]) and numerical studies on torsion and ductility demand of buildings [Sommer, 1999].

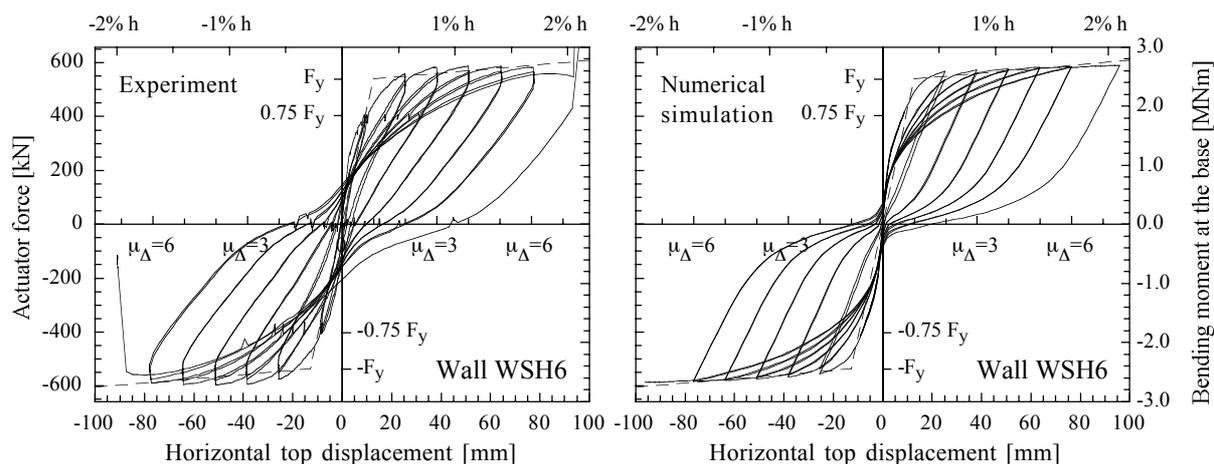
## EXPERIMENTAL INVESTIGATIONS

Different kinds of tests have been performed to investigate the behaviour of RC structural walls. In this contribution only the cyclic-static tests can be briefly presented. An extensive summary of all test results and all test data is available at the web site of the Institute of Structural Engineering at the ETH. Six reinforced concrete walls, designed by the capacity design method according to [NZS 3101, 1995] and [Paulay & Priestley, 1992], were tested under cyclic-static action. The specimens WSH1 through WSH6 represented the lower part of a reinforced concrete structural wall of the six-storey reference building shown on the left in Figure 1 and were scaled at 1:2. The test set-up (Figure 2 left) reproduced the same conditions for stress resultants in the plastic region of the specimen as in the structural walls of the reference building under seismic action (Figure 1 right). The base of each specimen was rigidly connected to the strong floor of the laboratory. The top of the wall was horizontally displaced back and forth in the plane of the wall by a hydraulic actuator. The first two cycles were force-controlled up to 75% of the calculated ideal bending strength to determine the idealised yield displacement,  $\Delta_y$ , and the displacement ductility  $\mu_\Delta=1$ . In the subsequent cycles, the displacement ductility was increased in steps of 1, beginning with  $\mu_\Delta=2$ . For each ductility step, two loading cycles were employed. The axial force due to gravity loads was applied by post-tensioning and held constant during the tests. The test parameters included the ductility of the reinforcing steel, the reinforcement ratio, the axial force, and the design method. The behaviour of all test specimens was observed until failure. The instrumentation permitted the measurement of the acting forces, the horizontal and vertical displacements, the strains along the corner reinforcement, the strain field of the plastic region, and the crack widths.



**Figure 2: Front view of the set-up for the cyclic-static tests with wall specimen WSH6 after failure (left) and reinforcement of the plastic region of wall specimen WSH6 (right, dimensions in mm)**

The lateral force vs. displacement curves of the specimen are among the most significant results as they show the dominant influence of the selected parameters, that of specimen WSH6 is shown on the left in Figure 3. Furthermore, the test showed the improvement of the seismic behaviour ensured by capacity design, the influence of a high axial force on the failure pattern and energy dissipation capacity of structural walls. The mechanical properties of the reinforcement strongly affected the stability of the rebars in the compression zone and the extent of yielding in the tension zone, so that related design provisions have to be refined. Crack pattern, axial force and shear deformations, the latter not being negligible even if shear reinforcement remains elastic, strongly affects the yield displacement. Shear deformation increases with increasing displacement ductility despite an essentially constant shear demand. Detailed results of the cyclic-static tests and the consequences on the design of structural wall systems will be presented in [Dazio, 2000].

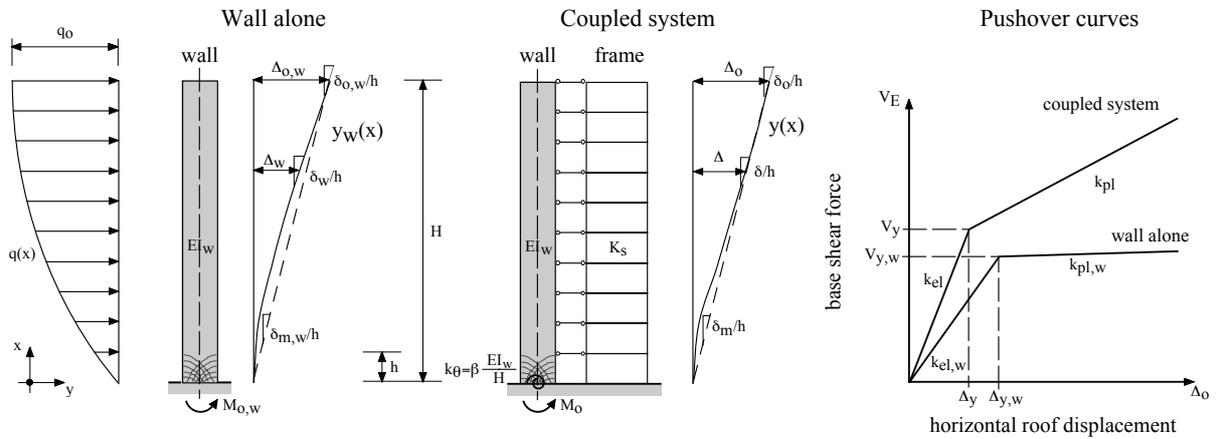


**Figure 3: Hysteretic behaviour of wall specimen WSH6, test (left) and numerical simulation (right)**

### SIMPLIFIED ANALYTICAL MODEL

Capacity-designed structural wall systems exhibit a single and invariant plastic mechanism and its response is not severely affected by higher modes effects. For these reasons pushover analysis, even if it has many limitations, seems to be a very efficient tool to assess the nonlinear behaviour of such structures. In the following

a simplified analytical method to perform a hand calculation of a bilinear approximation of the pushover curve of structural wall systems is presented. In this contribution only the calculation of the pushover curve is explained. A more comprehensive design procedure, also based on the findings shown in [Bachmann & Dazio, 1997] will be presented in [Dazio, 2000]. The pushover curve calculated by hand can be used, for example, in combination with the capacity spectrum method ([Fajfar, 1998]), the N2-method ([Fajfar et al., 1996]) or with a simple time-history analysis on a substitute SDOF-System during a preliminary calculation or to check a more refined pushover curve resulting from a nonlinear static computer analysis. In structural wall systems the main part of the stiffness resisting horizontal forces stems in general from the structural walls acting as cantilever flexural beam. However, the contribution of the frame action of the slabs, structural walls and columns forming together a cantilever shear beam may be important. The contribution of the frame action increases, in proportion, when the plastic region of the structural walls starts to yield. For this reason structural wall systems can be modelled as a coupled system as shown in Figure 4, consisting of a flexural beam with the equivalent sectional stiffness  $EI_w$  continuously coupled with a shear beam with shear stiffness  $K_s$ . As long as the structural walls are elastic the flexural beam can be considered as fully fixed at the bottom. When the plastic regions at the base of the structural walls start yielding the flexural stiffness rapidly decreases. This phenomenon can be taken into account considering the flexural beam as only partially restrained by a flexural spring with rotational stiffness  $k_\theta$ .



**Figure 4: Simplified model to consider the interaction between walls and frames in a multistorey building**

According to [König & Liphardt, 1985] the differential equation governing the horizontal displacement  $y(x)$  of the coupled system can be written as

$$EI_w \cdot y^{iv}(x) - K_s \cdot y^{ii}(x) = q(x) \quad (1)$$

with the following boundary conditions for the more general case of a partially restrained flexural beam:

$$y(0) = 0 \quad , \quad k_\theta \cdot y^i(0) = EI_w \cdot y^{ii}(0) \quad , \quad y^{ii}(H) = 0 \quad , \quad EI_w \cdot y^{iii}(H) = K_s \cdot y^i(H) \quad (2)$$

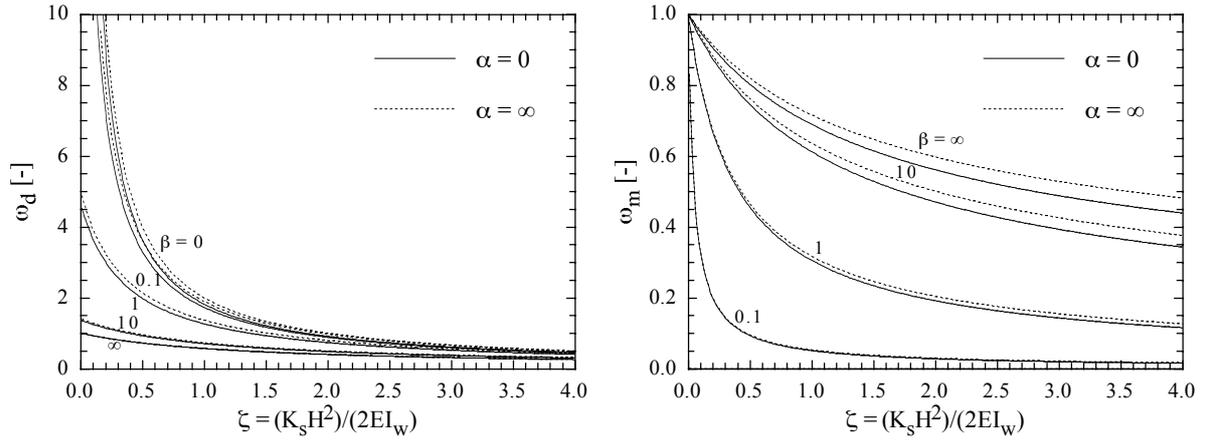
The distribution of the lateral force acting on the structural wall system can be described by means of the following general expression proposed by [Miranda, 1999]:

$$q(x) = q_o \cdot \frac{1 - e^{-\alpha x/H}}{1 - e^{-\alpha}} \quad (3)$$

A value  $\alpha=0$  corresponds to the common assumption of a triangular force distribution and a value  $\alpha=\infty$  corresponds to a uniform force distribution. Introducing the parameters

$$\zeta = \frac{K_s H^2}{2EI_w} \quad , \quad \beta = \frac{k_\theta H}{EI_w} \quad , \quad \omega_v = \frac{\Delta_o}{\Delta_{o,w}} \quad , \quad \omega_m = \frac{M_o}{M_{o,w}} \quad (4)$$

the differential equation (1) can be easily solved by means of standard mathematical software ([Maple, 1997]) giving the relationships shown in Figure 5.



**Figure 5: Parameter  $\omega_d$  to calculate the horizontal roof displacement  $\Delta_o$  of the coupled system (left) and parameter  $\omega_m$  to calculate the yield base shear  $V_y$  of the coupled system (right)**

The major difficulty of this simplified method is a suitable estimation of the equivalent sectional stiffness  $EI_w$ . A possible method is to consider a reduced sectional stiffness due to cracking using for example the recommendations of [NZS 3101, 1995]. A more promising procedure involves the calculation of the wall stiffness  $k_{el,w} = V_{y,w}/\Delta_{y,w}$  as proposed by [Priestley & Kowalsky, 1998] and derives the value of  $EI_w$  from  $k_{el,w}$ . An extensive parametric study should clarify this problem. The stiffness  $K_s$  of the shear beam can be estimated considering assemblages consisting of slabs and vertical elements as shown in [Bachmann & Dazio, 1997]. The stiffness  $k_\theta$  of the rotational spring can be calculated with  $\beta = \infty$  while the structural walls remain elastic and with  $\beta = 0 \dots 0.20$  when the plastic region of the structural walls yields. This corresponds to a ratio  $r_o$  of post-yield to pre-yield stiffness  $k_{pl,w}/k_{el,w}$  of the wall ranging between 0 and 5%. The bilinear approximation of the pushover curve can now be readily calculated using equations (5) to (7):

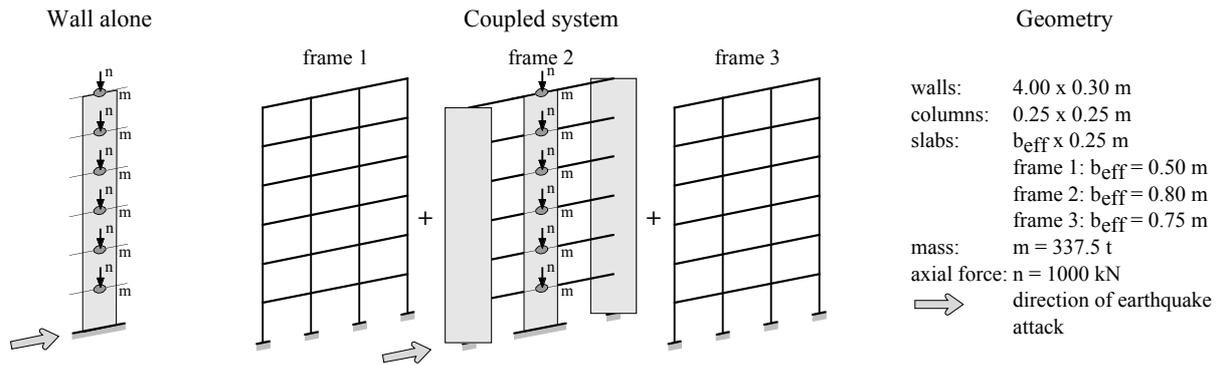
$$\Delta_o = \omega_d \cdot \frac{-24 + 3\alpha^4 e^\alpha - 24\alpha e^\alpha + 24e^\alpha + 12\alpha^2 + 8\alpha^3}{24\alpha^4 (e^\alpha - 1)} \cdot \frac{q_o H^4}{EI_w} = \omega_d \cdot \Delta_{o,w} \quad (5)$$

$$k = \frac{1}{\omega_d} \cdot \frac{\alpha^3 (\alpha e^\alpha + 1 - e^\alpha)}{-24 + 3\alpha^4 e^\alpha - 24\alpha e^\alpha + 24e^\alpha + 12\alpha^2 + 8\alpha^3} \cdot \frac{24EI_w}{H^3} = \frac{1}{\omega_d} \cdot k_w \quad (6)$$

$$V_y = \frac{1}{\omega_m} \cdot \frac{\alpha (\alpha e^\alpha + 1 - e^\alpha)}{\alpha^2 e^\alpha + 2\alpha - 2e^\alpha + 2} \cdot \frac{2M_{y,w}}{H} = \frac{1}{\omega_m} \cdot V_{y,w} \quad (7)$$

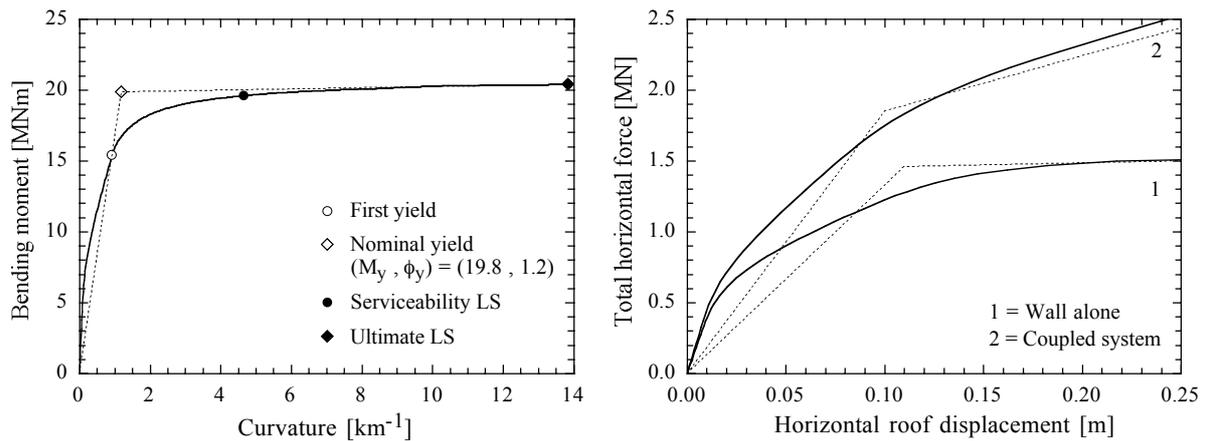
## NUMERICAL INVESTIGATIONS

In order to evaluate the performance of the simplified analytical method proposed in Section 3, some nonlinear numerical simulations were carried out using the self-developed software RECHENBRETT 2D ([Dazio, 2000]). In this, nonlinear structural elements are modelled with Bernoulli beam elements (cubic Hermitian interpolation functions) and the internal element forces are computed by means of a fibre model. The simulations were performed on the six-storey reference building shown in Figure 1 considering an earthquake attack in the transverse direction. The building is nearly symmetric so that only one half was modelled in order to reduce the computational effort. The wall specimen WSH6 corresponds in scale 1:2 to the prototype structural wall which restrains the reference building in the transverse direction. During the first stage the reliability of the chosen numerical model was checked comparing the experimental and the numerical hysteretic behaviour of specimen WSH6. Figure 3 shows a fairly good agreement between the two curves. Afterwards the cross-section of the prototype structural wall was definitely defined by scaling that used to simulate specimen WSH6 by a factor 2. The moment-curvature of the prototype structural wall is shown on the left in Figure 7, the nominal yield values are calculated according to [Priestley & Kowalsky, 1998] and are  $M_y = 19.4 \text{ MNm}$ ,  $\phi_y = 1.2 \times 10^{-3} \text{ m}^{-1}$ , respectively. Both the static and dynamic nonlinear analyses were performed on two different discretizations of the reference building as shown in Figure 6. In the first discretization only the structural wall is modelled while in the second also three frames resulting from the interaction of columns, slabs and structural walls are considered. The assumptions made regarding the geometry of the frame elements are listed in Figure 6.



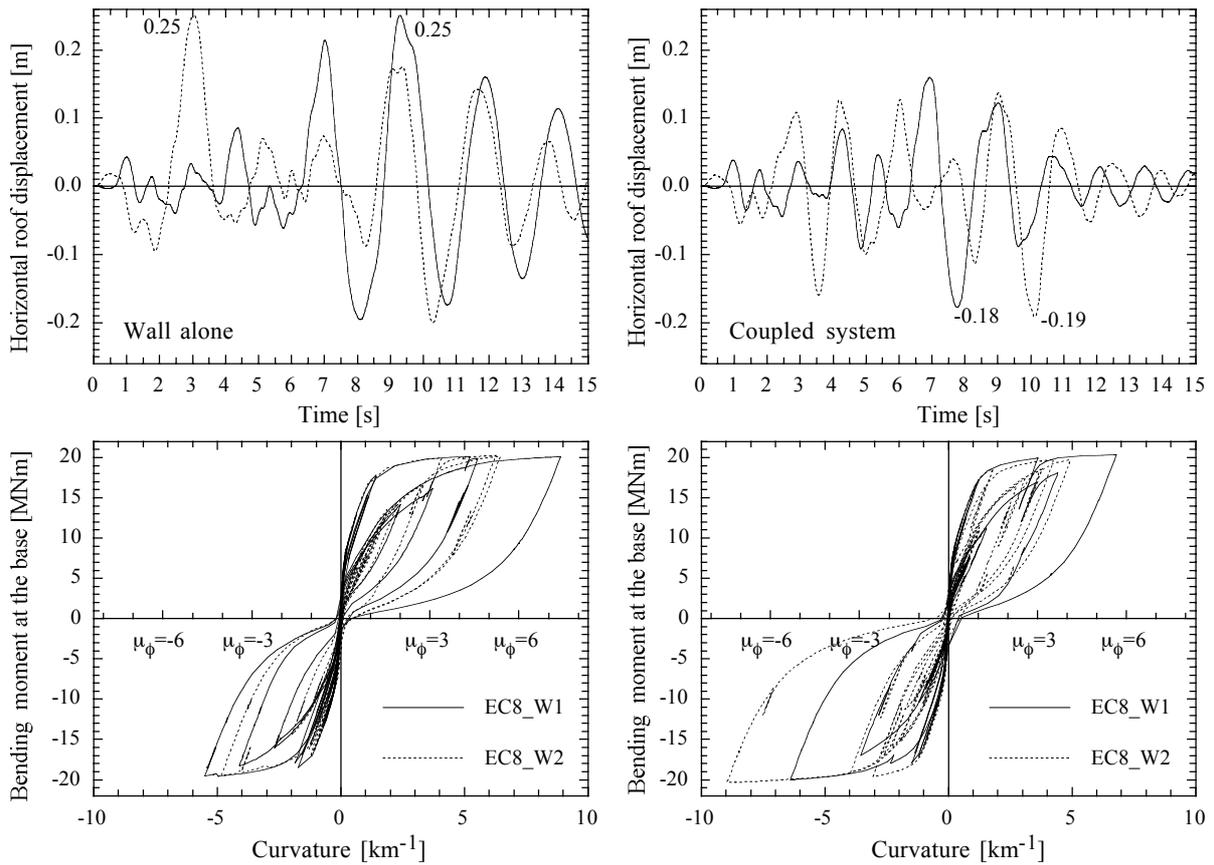
**Figure 6: Discretisation of the reference building shown in Figure 1 to perform the numerical simulations**

The static analyses giving the pushover curves shown on the right in Figure 7 were carried out assuming a triangular force distribution. The latter was kept constant during the analysis while the magnitude of the force was stepwise increased. On the right in Figure 7 the pushover curves computed by means of nonlinear computer analyses are drawn with solid lines and can be compared with the hand calculations carried out using the simplified method proposed in Section 3 and shown as dotted lines. For the hand calculation the following assumptions were made:  $EI_w=21000 \text{ MNm}^2 \approx 0.35EI_g$ ,  $K_s=48 \text{ MN}$  and  $\beta=0.10$  when the structural wall starts yielding. Without any post-calibration of the calculations the simplified hand calculations give a good description of the computed curves.

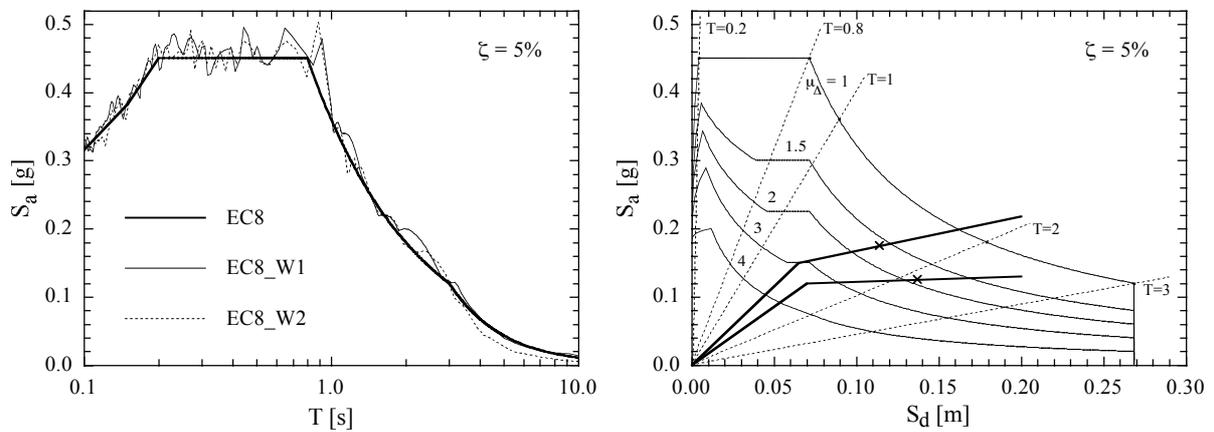


**Figure 7: Moment-curvature relationship for the prototype wall (left). Pushover curve of both “wall alone” and “coupled system” (right), the solid lines indicate the nonlinear static computer analyses and the dotted lines indicate the hand calculations according to Section 3**

The time-history analyses were carried out considering two different artificial earthquakes compatible to the elastic design response-spectrum prescribed by [Eurocode 8, 1997] in the case of soft soil and a design ground acceleration  $a_g=0.20g$ . The response-spectra of the two earthquakes “EC8\_W1” and “EC8\_W2” are shown on the left in Figure 9. About 5% additional damping accounting the contribution of non-structural elements was considered. The fundamental mode was damped by the introduction of a mass proportional damping matrix  $C=a_0M$  while the higher modes were damped with numerical damping provided by setting  $\alpha=-0.33$  in the well known time integration “Hilber-Hughes-Taylor  $\alpha$ -algorithm” implemented in RECHENBRETT 2D. The results of the time-history analyses are shown in Figure 8. The horizontal roof displacement in the case of the wall alone is about 35% greater than in the case of the coupled system, and also the period is greater in the first case. On the other hand the ductility demand in the plastic zone of the structural wall seems to be similar in both cases. Theoretical considerations on the coupled system of Figure 4 show that by equal roof displacement, the bending moment at the base of the flexural beam increases with increasing frame action. This latter statement confirms the results of the time-history analysis. In order to show a possible application of the pushover curves calculated by hand, the expected roof displacement of the reference building discretized as both “wall alone” and “coupled system” subjected to an earthquake compatible to Eurocode 8 “soft soil” are estimated using the capacity spectrum method and compared with the results of the time-history analysis.



**Figure 8: Results of the nonlinear time-history analyses of the prototype building submitted to two different earthquakes compatible to Eurocode 8 “soft soil”, EC8\_W1 and EC8\_W2. Time-history of the roof displacement (above) and moment-curvature relationship at the base of the prototype wall (below)**



**Figure 9: Spectra of the 2 artificial earthquakes compatible to EC8 “soft soil” used in the numerical simulation (left). Demand and capacity spectra for the “wall alone” and for the “coupled system” (right)**

The pushover curve has first to be transformed to a force-deformation and subsequently to an acceleration-displacement relationship (capacity curve) of an equivalent SDOF system. The latter curve is then plotted in the demand spectrum of the earthquake as shown on the right in Figure 9, and the displacement demand of the equivalent SDOF system can be determined. In this example it amounts to approx. 13.5 cm in the case of the “wall alone” and to approx. 11.3 cm in the case of the “coupled system”. The displacement demand of the equivalent SDOF system can now be transformed back to the roof displacement of the reference building by multiplication with a factor  $\Gamma=1.55$ . The values of 20.9 cm and 17.5 cm agree rather well with the results of the time-history analysis shown in Figure 8.

## CONCLUSIONS

Structural wall systems consisting of flat slabs, small columns designed for gravity loads only and a few relatively slender structural walls of rectangular cross-section are popular in Switzerland and central Europe and can easily be designed for a ductile behaviour under moderate earthquake actions. The experimental investigations performed at the ETH represent a suitable basis to adapt proven design rules for central Europe with moderate seismicity pertaining to structural walls found in countries with high seismicity. The presented analytical method to calculate by hand a bilinear approximation of the pushover curve of regular structural wall systems has a reliable theoretical background and leads to promising results. An extensive parametric study should investigate the limitations of the method and clarify the determination of the nominal yield displacement. The developed computer code RECHENBRETT 2D has limited options but its capability to check the response of cyclic-static and dynamic tests and to predict the response of the pseudodynamic tests, all performed at the ETH, make it a useful and reliable tool to investigate the nonlinear behaviour, both static and dynamic, of regular structural wall systems.

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