SEISMIC VULNERABILITY OF OLDER SWISS R. C. BUILDINGS

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SUMMARY

The importance of the seismic risk has only recently been recognised in Switzerland, and older buildings were built without consideration of a potential seismic impact. Older Swiss r.c. buildings form an important part of the building inventory and their contribution to the seismic risk is potentially large. This paper presents findings of a research project conducted to further the knowledge of the seismic vulnerability of Swiss reinforced concrete multistory buildings constructed between 1945 and 1989. The dominant structural types are identified and expected material properties are discussed. Based on a field survey, example buildings were selected and analysed with different analysis methods. The comparison of the analysis results shows the value of nonlinear analysis methods for realistic seismic evaluations.

INTRODUCTION

Though seismicity in Switzerland is only medium to low, there is a need to investigate the seismic performance of older structures. About one third of all existing buildings were constructed in reinforced concrete without consideration of seismic loading. Effective seismic code regulations were introduced in 1989. An unknown number of seismically vulnerable structures exist and a few vulnerable structures have been retrofitted. Scenario studies revealed the big impact of a possible repeat of historical earthquakes, such as the 1356 Basel earthquake. The seismic exposure is large in comparison to other natural hazards which have traditionally been given more attention in Switzerland [Katanos, 1995] [Bachmann, 1998a].

This paper presents findings of a research project underway at the Swiss Federal Institute of Technology in Lausanne (EPFL), Switzerland. Its aim is to investigate the use of a nonlinear static analysis procedure for the seismic evaluation of older r.c. buildings with bearing walls, which are common in Switzerland. This project is a part of an international effort to better assess the actual seismic vulnerability of older structures (e.g. [Moehle, 1998]). Because of the very high cost of retrofitting and even higher cost of failure of buildings during seismic events, the realistic assessment of seismically vulnerable structures is very important economically. For the purpose of this study, a survey of older r.c. buildings in two Swiss towns was conducted and the construction techniques of these older buildings were documented. Representative buildings were selected, and their seismic vulnerability was evaluated through detailed analyses.

SEISMIC PERFORMANCE

Considerable advances have been achieved in the last decades in the seismic design area [Bertero, 1996]. Modern design concepts aim at minimisation of the building's earthquake risk composed of building damages and casualties. The concept of "performance based design" has gained much recognition [SEAOC, 1996]. This study of the seismic vulnerability of older structures is conducted within the conceptual framework of performance based design philosophy. The following parameters are considered as controlling the seismic performance: the lateral structural resistance, the lateral deformation capacity (including stiffness, ductility and cyclic energy dissipation) and the response of non-structural elements.

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OLDER SWISS R.C. BUILDINGS

The buildings of interest are multistory reinforced concrete structures built in the period from 1945 to 1989. The first date is at the beginning of an intensive construction activity, while the second date indicates the introduction of seismic regulations into the Swiss building codes. In 1970, the seismic regulations specified that buildings have to resist lateral forces equal to 2% of their weight (important buildings 2.8%) while in 1989 more stringent seismic regulations were introduced prescribing much higher seismic loading (see table 1).

To gather information on older r.c. buildings, several sources were exploited. A survey in two Swiss towns with medium seismicity was conducted providing basic information. Over 50 buildings were selected because of their construction date and their potential seismic vulnerability. Construction plans could be found, but the structural design calculations could only be found for a few of the buildings.

The survey confirmed that r.c. bearing walls are predominant in existing Swiss buildings and that frame-type structural systems are an exception. Basements with massive r.c. walls are another typical element of Swiss buildings, providing stiff support for the bearing walls. The walls and the slabs are built of cast-in-place reinforced concrete. Precast reinforced concrete elements are in some cases used for non-structural elements. In the highest floors, where the gravity loading is limited, structural masonry walls sometimes take the place of r.c. walls.

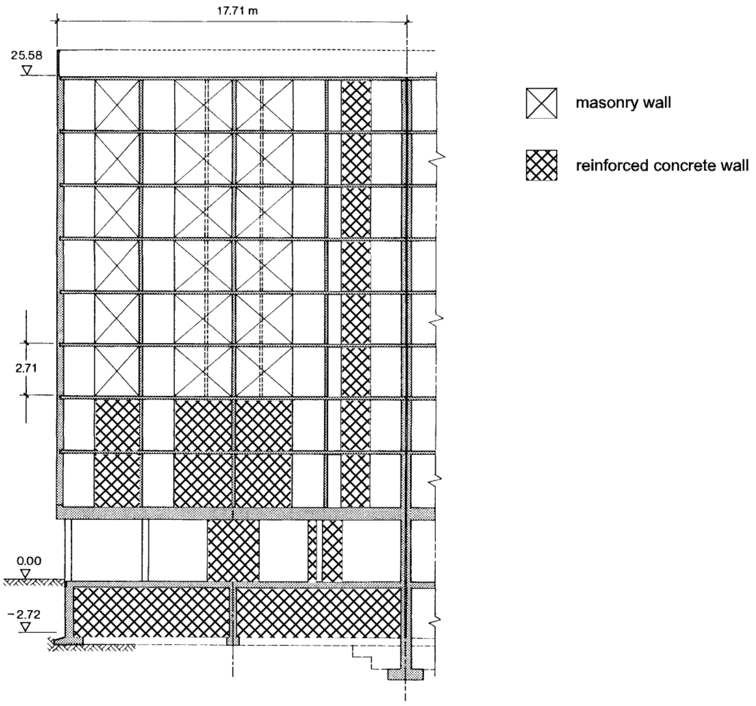

Figure 1 : Longitudinal section of the example building KJA

Most buildings of the study period are of one of the following two structural types. The first type is characterised by the fact that the partition walls are structural bearing walls. Because there are many walls, the lateral resistance of such buildings is generally high. The second structural type is similar to frame structures with the difference that the elevator core and the stairs are enclosed in structural walls. Columns are designed for gravity loading only. While the first structural type is common for apartment buildings, the second one is often found for office buildings. In both types, structural walls provide substantial lateral resistance and stiffness, limiting interstory drifts. In the following, two example buildings representative of this second structural type are described and their seismic vulnerability in the longitudinal direction is discussed.

The KJA example building is a 9 story apartment building, 53.1 m long and 13.6 m wide (figure 1). It is separated into three blocks by two expansion joints. The basement contains massive walls founded on strip footings. The building has a soft ground floor. The columns take a big part of the gravity loading while the r.c.

walls take the horizontal loading. The walls are part of the elevator core or the stair shaft. In the highest six stories, some masonry walls replace r.c. walls. The lateral resistance in the longitudinal direction is provided primarily by the walls shown in figure 1.

The second building is the PMS building, a 5-story office building, 23.4 m long and 15.5 m wide. Its lateral resistance in the longitudinal direction is provided by a central elevator core and two frames built in the building facade. The frames are constituted by two columns and edge beams.

DESIGN AND CONSTRUCTION OF OLDER SWISS R.C. BUILDINGS

The evaluation of older structures requires a good understanding of the design codes according to which they were designed. From 1945 till 1989 three different code generations were used in Switzerland. Under gravity loading, the requirements for the structural safety of common buildings remained about the same. This was verified by comparison of design examples. No direct comparison of the codes is possible because the verification criteria changed from an allowable stress concept to a verification of the ultimate resistance. The horizontal loading, however, increased strongly as shown in table 1. Loading with a rare occurrence like maximum snow actions or earthquake actions increased substantially.

Table 1 : Global horizontal forces acting on building structures, according Swiss building codes [SIA, 1970, 1989 & 1994].

code regulations [SIA 1970, 1989 & 1994]		example building KJA base shear in [kN]	example building PMS base shear in [kN]
1935 – 1956	Wind	212	172
	Earthquake	none	none
1956 – 1970	Wind	378	306
	Earthquake	none	none
1970 – 1989	Wind	378	306
	Earthquake	484	388
since 1989	Wind	290	235
	Earthquake (reduced for nonlinear response)	1651	1059

Table 1 presents the unfactored design lateral loads according to the building codes valid in different periods for the two example buildings,. The loads are given for the longitudinal direction, where in both buildings the lateral resistance is lower. The earthquake loads given contain the reductions for inelastic response, i.e. they can directly compare with the wind loads. Evidently, the introduction of the seismic regulations in 1989 increased the required lateral load resistance substantially.

In order to evaluate the cyclic response of older r.c. structures, the material properties of older reinforcements, typical of the study period were documented. In Switzerland, cold formed high strength reinforcement was available at affordable prices. Its development and implementation was supported by the research of the Swiss Federal Laboratories for Material Testing and Research (EMPA). From 1940 till 1950, EMPA conducted over 10 test series with reinforced concrete elements using 8 different reinforcement steels [EMPA, 1950]. The test report includes the behaviour of reinforced concrete elements up to the ultimate response and the plastic behaviour. Figure 2 shows rebars of the cold formed steel "Tor 40" produced in Switzerland. After 1950, EMPA continued to regularly test the reinforcement steels produced in Switzerland.

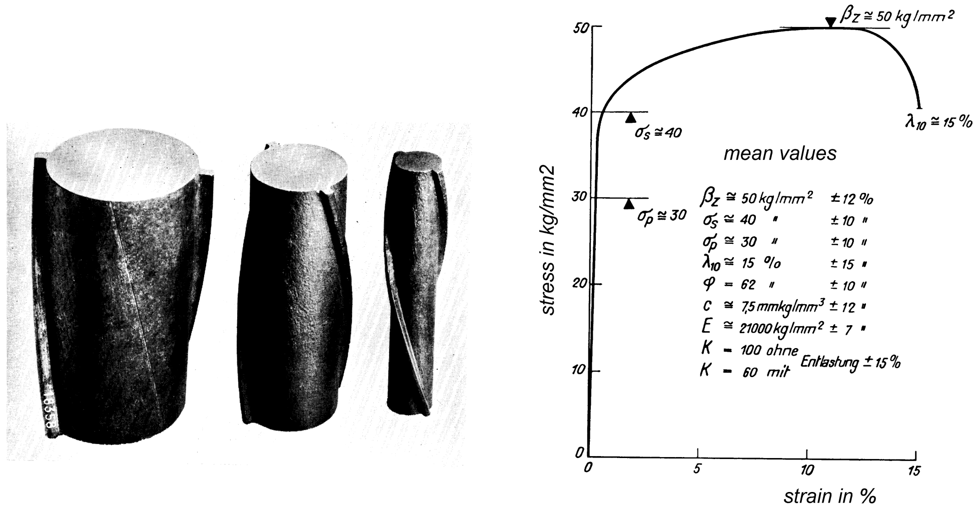

Figure 2 : Cold formed reinforcement rebars "Tor 40" common in Swiss r.c. constructions in the 1950's [EMPA, 1950].

Table 2 gives a rough overview on the reinforcement properties used in Swiss r.c. constructions from 1950 to 1989. For structural purposes, from 1950 on, mainly high strength reinforcement was used while mild steel was used for confinements or constructive reinforcement. The reinforcement properties important to ensure a satisfying ductility of r.c. structures are the ratio of tensile strength to yield strength ($R_m/R_{p\ 0.2}$) and the total strain at maximum rebar force (A_{gt}), as described in [Bachmann, 1998b]. Bachmann recommends the following values to guarantee a substantial plastic deformation capacity: $R_m/R_{p\ 0.2} \geq 1.15$ and $A_{gt} \geq 0.06$. Table 2 shows that in most cases the high strength reinforcement used in older Swiss r.c. buildings satisfy these requirements.

Table 2 : Properties of the most important reinforcement steels in Swiss r.c. construction according to codes and tests [EMPA, 1950], [SIA, 1994].

period	product	product description	nominal properties according to codes or separate regulations			probable mean values of properties, based on test results and company's indications		
			$R_{p\ 0.2}$ [MPa]	R_m [MPa]	A_{gt} [-]	$R_{p\ 0.2}$ [MPa]	R_m [MPa]	A_{gt} [-]
1956 – 1968	steel I	mild steel	240	360				
	steel IIa	cold formed steel, e.g. Tor 40, Caron	350	420		400 – 500	460 – 560	0.04 – 0.15
	steel IIb	"alloyed steel" e.g. Box-steel	350	520		350 – 500	550 – 700	0.15 – 0.20
1968 – 1989	steel I	mild steel	240	430				
	steel IIIa	"alloyed steel"	430	560				
	steel IIIb	cold formed steel, e.g. Torip, Roll-S	430	480		540 (≥500)	630 (≥560)	~0.07
	steel IV	welded wire fabric	540	570				
	tempered steel	e.g. Topar 500S				550 (≥500)	640	~0.10
1989 – today	S500a, b, c		460	550 – 600	0.05 – 0.15			

$R_{p\ 0.2}$ yield strength $\qquad R_m$ tensile strength
A_{gt} strain at maximum rebar force, not contained in Swiss building codes, but estimated based on the uniform rebar elongation aside the rebar fracture

Code requirements for concrete have been studied, too. The required concrete resistance was somewhat lower than today. As concrete tends to become more resistant with age, the authors assumed for the example buildings concrete properties identical with the properties of nowadays mostly used concrete (mean compressive strength of 35 MPa). Besides the safety verifications, few written guidelines have been found for the construction and detail design of r.c. structures. As workmanship was less expensive than material, the reinforcement was set in place with care for details. The thinnest r.c. walls found in existing buildings are 140 mm thick. Slabs are 180 mm and up. The reinforcement of r.c. walls according to studied construction plans is between 0.2 – 0.4% in horizontal direction and between 0.4 – 0.7% in vertical direction. The edges of walls are always confined with u-shaped stirrups. The overlap of the reinforcement was equal or greater than 40 rebar diameters and till 1968 the anchorage of reinforcement rebars was most often realised with a hook. The cover of the rebar had to be 20 mm for exposed structural elements.

The findings regarding material properties and constructive detailing show that the basic conditions necessary for a ductile response of reinforced concrete are met. That r.c. walls in older buildings can have a substantial energy dissipation capacity has been confirmed experimentally. R.C. walls were tested on a shaking table [Lestuzzi, 1999] at the Swiss Federal Institute Zürich. One of the walls was designed according to conventional design criteria, and like older walls, showed a substantial energy dissipation capacity .

SEISMIC ANALYSIS USING DIFFERENT ANALYSIS METHODS

The choice of the analysis method(s) is central to the seismic evaluation of existing buildings. In Switzerland it is likely that a practising engineer would start with one of two available design code procedures. One procedure is given by the building code valuable since 1989 [SIA 160, 1989] and the other is specified in the national application document of the European prestandard ENV 1998 [SIA, 1997], applicable since 1997. Both are based on linear elastic structural analysis. If no clearly sufficient seismic resistance can be proved with a simple method, a more realistic seismic analysis is advisable. Such more realistic analysis can be conducted with nonlinear dynamic analyses (NLDA) or a nonlinear static procedure, e.g. the Capacity Spectrum Method (CSM). In the USA, the CSM is recognised as an efficient and reliable analysis tool for frame buildings [Freeman, 1998], [ATC-40, 1996], [FEMA 273, 1997]. That the CSM is a useful tool for bearing wall buildings is shown in the companion paper [Peter, 2000], where a short description of the CSM is given. The value of the CSM for the graphical evaluation of retrofit measures is illustrated in [Badoux, 2000].

For comparison of the analysis methods, the response of the two example buildings (described above) to a given seismic input was analysed with four different methods. The ENV 1998 response spectrum for soil class B and a peak ground acceleration of 0.16g was chosen (figure 3). Many buildings in Switzerland are founded on a class B soil while the peak ground acceleration stands for the Swiss seismic zone with the highest seismicity. An artificial time-history, taken from [Lestuzzi, 1999], has been modified to meet the ENV 1998 response spectrum. For the nonlinear analyses, the software Idarc2d, version 5 [Valles, 1996] was used. It had previously been adapted and calibrated for bearing wall computation [Peter, 2000].

The seismic analysis with the CSM is illustrated in figure 3. A substantial nonlinear response is observed for both buildings. With the assumption of an equivalent damping of 5%, a spectral displacement of S_d = 50 mm is estimated for PMS building and of S_d = 78 mm for KJA building. The application of the NLDA to the PMS building is illustrated in figure 4. At the maximum overall deformation state, a spectral displacement of S_d = 40 mm was computed. For the KJA building, a maximum S_d = 48 mm was calculated.

The seismic response of the example buildings was then analysed with the two code procedures. The internal seismic forces were computed using the same building models established for the CSM, but with the elastic response only. The same structural resistance, based on mean material properties, is used for all analyses. The horizontal force procedure given in the building code SIA 160 [SIA 160, 1989] to compute equivalent lateral forces similar to procedures found in design codes of many countries [IAEE, 1996]. The total horizontal forces are computed with the total building mass (m_{tot}), a spectral acceleration (a_h) corresponding with the spectral acceleration for the fundamental mode and a factor accounting for nonlinear response (K): $Q_{acc} = m_{tot} \cdot a_h \cdot C_d / K$ (here C_d = 1.0). Up to 25% of the seismic actions can be redistributed between structural elements.

For the computation of the ENV 1998 seismic loading, the elastic response spectrum analysis was performed. To account for nonlinear effects, the elastic response spectrum shown in figure 3 was modified: The elastic spectral accelerations were reduced with the behaviour factor q and the spectrum's shape was adapted. The model uses section stiffness based on the uncracked concrete section. The internal modal forces are combined using the square root of the sum of the squares (SRSS). For independent walls, a maximum of 30% of the internal forces can be redistributed (ductility classes DC "L" and "M").

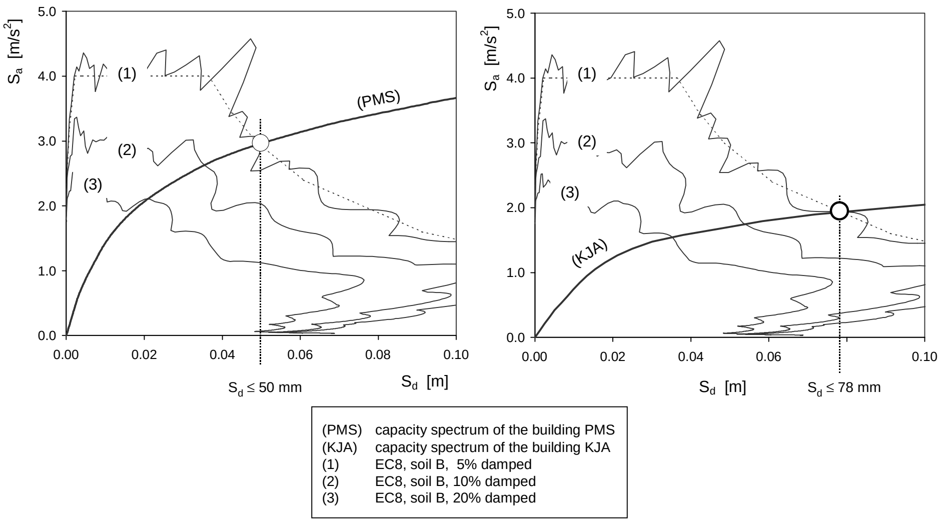

Figure 3 : Capacity spectrum method applied on the example buildings. The spectral target displacement is here determined with the 5% damped response spectrum yielding a spectral target displacement of S_d = 50 mm for the building PMS and S_d = 78 mm for the building KJA.

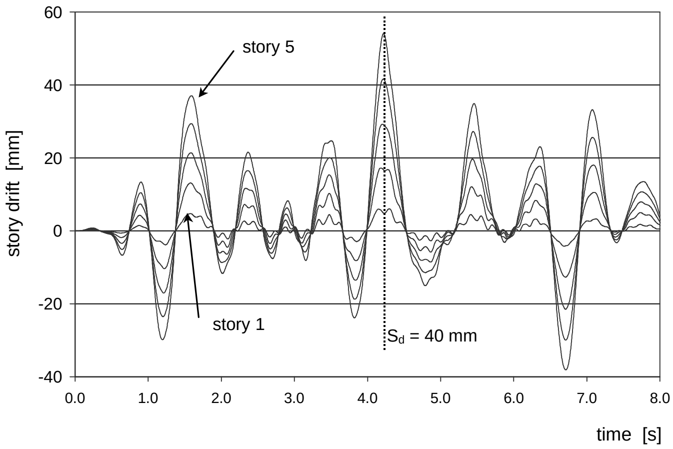

Figure 4 : Story drifts in the nonlinear dynamic analysis (NLDA) of the PMS building for the artificial acceleration time-history representing the ENV 1998 response spectrum.

The most important analysis results are summarised in table 3. They confirm that the CSM estimates the seismic response realistically, compared with the results of the NLDA [Peter 2000]. The two linear methods however tend to underestimate the seismic capacity and they do not correctly predict the failure mechanism.

Table 3 : Seismic evaluation of two example buildings, PMS and KJA, with different methods.

seismic loading according to ENV 1998: 0.16g peak ground acceleration, soil class B		PMS building (longitudinal direction)	KJA building (longitudinal direction)
model eigenfrequencies		$f_1 = 2.0$ Hz, $f_2 = 8.0$ Hz	$f_1 = 1.4$ Hz, $f_2 = 5.0$ Hz
Capacity Spectrum Method (CSM)	spectral displacement S_d (5% equivalent damping)	50 mm	78 mm
	base shear coefficient V/W	0.21	0.11
	evaluation result	some structural damage (yielding of some columns in higher stories up to max. 40% of their deformation capacity)	failure of 1 structural wall, important structural damage (sufficient resistance in ground floor)
nonlinear dynamic analysis (NLDA)	spectral displacement S_d	40 mm	48 mm
	evaluation result	little structural damage (yielding of some columns in upper floors up to max. 30% of their deformation capacity)	failure of 4 structural walls (sufficient resistance in ground floor)
Horizontal lateral force procedure [SIA 160, 1989]	deformation coefficient K	2.5	2.0
	base shear coefficient V/W (reduced with K)	0.16	0.17
	evaluation result	insufficient resistance (excess shear demand in ground floor)	insufficient resistance (insufficient resistance in ground floor)
Response spectrum analysis according ENV 1998 [SIA, 1997]	behaviour factor q	2.5	1.6
	base shear coefficient V/W (reduced with q)	0.12	0.13
	evaluation result	substantial structural damage (resistance equals inelastic seismic efforts in the strongest wall)	insufficient resistance (insufficient resistance in ground floor)

CONCLUSIONS

A survey of representative building populations, the investigation of former construction techniques, and the evaluation of two example buildings with different analysis methods lead to the following findings regarding the seismic vulnerability of older Swiss multistory reinforced concrete buildings.

- The survey confirmed that buildings with with r.c. bearing walls are dominant and that frame structures are an exception. The buildings therefore have of a substantial lateral resistance and interstory drifts are limited. Quite often the bearing walls are reinforced concrete in the lower stories and masonry in the upper stories. These masonry walls can constitute "weak links".
- It is expected that r.c. walls in older Swiss buildings would generally display substantial seismic energy dissipation capacity. This is based on the study of construction design and detailing during the study period, as well as available experimental tests [Lestuzzi, 1999].
- The nonlinear static Capacity Spectrum Method (CSM) is more realistic than available design code methods for the assessment of the seismic response and the detection of the probable failure mode.
- Even when they were not designed for seismic loading, r.c. buildings with bearing walls tend to display relatively high seismic resistance. This finding is in agreement with observations of the seismic performance of bearing wall buildings in many earthquakes [Fintel, 1995].

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