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Seismic damage model for regular structures

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SUMMARY

In this paper seismic damage ratio model of regular structures is analyzed. It is based on the following assumptions:

- *Seismic response of the regular structures (symmetric plans and constant vertical stiffness) can be well interpreted by using SDOF system as a mathematical model of the structures.*
- *Response parameters of the structure as ductility, stiffness change, energy balance and the number of plastic excursions can describe a real level of structural damage.*

Based on some known damage models, the original formula for damage ratio is given. The valorisation of these assumptions and proposed formula for damage ratio was done by comparing the results of experiment CAMUS3, done by Camus working group, in TRM-ECOEST 2 Research programme in EMSI Sacley, France.

***Key words**: seismic damage model, regular structures, seismic resistance determination.*

1. BACKGROUND

The procedure of structural seismic resistance determination contains three steps: Modelling, Analysis and Valorisation.

Chung [1] determines "damage" as a level of physical degradation with precise defined consequences to residual capacity of resistance and deformations. The failure is a specific level of damage without any capacity of resistance and deformations. Usually, in literature, the problem of structural damage is solved by calculation coefficient called Damage ratio (DR), calculated for structural element (partial) for the whole structure (global).

Damage ratios in various models are based on maximum values of structural response parameters or cumulative values and summing non-liner deformation cycles. For example:

- Park & Ang [2] model defines damage ratio as linear combination of plastic deformation (ductility) and energy dissipation.
- Hwwang & Scribner [3] model contains Gosain's energy index which is normalized by dissipated energy, stiffness and maximum displacement in the i^{th} cycle, also with initial stiffness, yield displacement and a number of cycles in which are $Pi>0,75\ Py$.
- Mizuhata & Nishigaki [3] model similarly as Park's model defines damage ratio as linear combination of plastic deformation (ductility) and energy dissipation as the result of maximum deformation, failure deformation under monotonic load, number of real cycles with specific deformation and the number of cycles which leads to the failure. Park's model uses coefficient $ß$ as an increment of dissipated energy resulted by cycle failure, and a limit deformation as ductility.

Structure modelling is the most important phase in dynamic problem design. Structural model and type of analysis are directly connected and it is impossible to treat them separately. The structural model has to satisfy two requirements: The real interpretation of structural dynamic properties: periods and shapes and it has to be in function of study aims. The relation between dynamic properties of structure and dominant earthquake frequencies determines dynamic amplifications of structural loads. The levels of earthquake analysis errors depend on at the least the precise phase of calculation. It is usually the earthquake ground motion. It means that the most sophisticated model of structure does not mean absolute correct calculation. That is the reason which justifies simplifications in structure modelling. Fajfar [4] describes the structure by two combined models:

Detailed model is used for the analysis of stiffness matrix and dynamic properties of the structure. Transformation of detailed model by condensation process defines simple SDOF model of the structure accurate for non-linear dynamic time-history analysis of structural response.

It is quite clear that the modelling of the structure and damage ratio analysis are fields well developed in earthquake engineering literature, and that these knowledge should be used and combined. The missing field is the connection between theoretical damage function and numerical calculation's results.

Intention of this paper is to connect calculation's results with the damage model and to declare damage ratio as a deterministic measure of structure's damage state after the earthquake.

2. DAMAGE RATIO MODEL FOR REGULAR STRUCTURES

Regular structure is a plane symmetrical building with constant vertical stiffness.

Seismic response of structures is possible to interpret by dynamic, time-history calculation, where the earthquake is given as digital time function of ground acceleration.

The regular structure can be modelled as SDOF system with the following structures parameters: weight (W), elastic stiffness ($K_{el.}=(BS)_y/u^y$), elasticity limit base shear $(BS)_y$, damping (x) and post-elastic behaviour (idealized hysteresis model). Index (y) declares damage starts point. All mentioned parameters are in function of structure material and failure mechanism.

Based on the in background chapter mentioned damage models, new original deterministic declaration for the damage ratio is analyzed.

The level of structural damage (damage ratio DR) can be described by the combination of the following calculated structure response parameters:

1. Displacement ductility (D) defines the measure of post-elastic region in which structure was during the earthquake.
2. Maximum base shear force $(BS)_{max}$ and maximum top displacement u_{max} define residual stiffness (K') of the structure at the end of the earthquake.
3. Number of yield excursions (N_Y) and hysteresis energy (E_H) define post-elastic cyclic nature of damage ratio developing.

The first two parameters define damage mechanism under monotonic load and the third one takes into account cyclic failure. Similar as in the mentioned damage models, damage coefficient ratio (DR) is defined as linear combination of these two groups, as follows:

$$DR = \frac{1}{30}\left[D + DK + \sqrt[3]{\left(N_Y E_H / W\right)}\right] \qquad (1)$$

where:

- $D=u^{max}/u^y$ is displacement ductility demand,
- $DK=K_{el.}/K'$ is relative degradation of stiffness at the end of earthquake,
  $K_{el.}=(BS)_y/u^y$ is initial structure stiffness,
  $K'=(BS)_{max}/u^{max}$ is residual secant stiffness of structure after the earthquake,
- (N_Y) is number of yield excursions reached during the earthquake,
- E_H/W is hysteresis energy per unit of structure mass, dissipated during the earthquake.

Damage ratio is a linear combination of plastic deformations, stiffness degradation and energy dissipation of structure during the earthquake time.

It's main function in the damage model is to describe condition of structure after earthquake. In such approach value of damage ratio can be used in two directions:

1. To declare decreased residual seismic resistance and increased residual damping coefficient of structure using the following two formulas (Figures 1 and 2):

$$S_Y^{RESIDUAL} = S_Y^{INITIAL} \cdot \sqrt{(1 - DR)} \qquad (2)$$

$$x^{RESIDUAL} = \frac{x^{INITIAL}}{\sqrt{(1 - DR)}} \qquad (3)$$

2. Implementation of damage ratio values (DR) in pre or post earthquake damage analysis.

That is solved by the connection damage ratio values (DR) with the values of damage level identification (S), defined in the Croatian codes for post disasters damage assessment.

These relations are presented in Table 1.

Table 1 Physical interpretation of damage coefficient

Damage ratio (DR)	*Structural damage description*	*Possibilities of technical and economic reparation*	*Code damage level (S) (1° to 6°)*
0 < DR < 0, 3	*insignificant*	*repairable*	*1° - 2°*
0,3 < DR < 0, 5	*moderate*	*repairable*	*3°*
0,5 < DR < 0, 8	*severe*	*repairable*	*4°*
0,8 < DR < 1,0	*heavy*	*repairable*	*5°*
1,0 < DR	*extremely high level or collapse*	*non-repairable*	*6°*

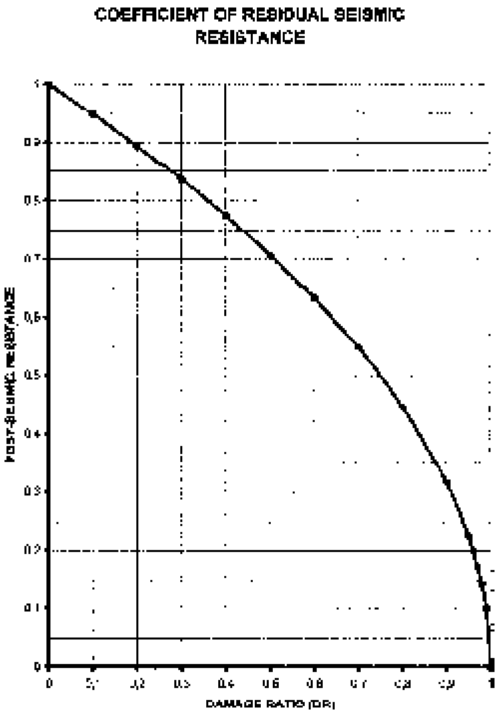


Fig. 1 Decrease of initial seismic resistance as a function of damage ratio (DR)

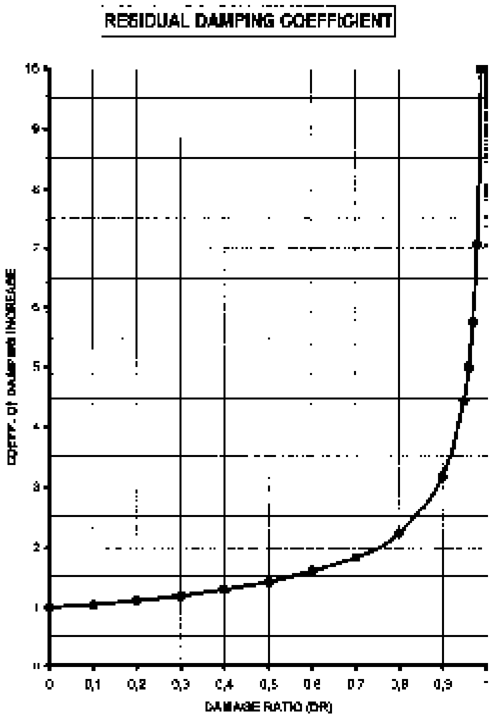


Fig. 2 Increase of initial damping coefficient as a function of damage ratio (DR)

3. EXPERIMENT SPECIMEN DESCRIPTION

Approach and damage model, described in previous chapter should be checked. Valorisation will be done by comparison with the results of experiment CAMUS3 done by Camus working group (July 2000.), TRM-ECOEST 2 Research programme in EMSI Sacley, France [5].

3.1 Geometrical description

The 1/3rd scaled model to be studied is composed of two parallel 5-floor R/C walls without opening, linked by 6 square floors (including the floor connected to the footing). A heavily reinforced concrete footing allows the anchorage to the shaking table.

The dimensions of the different parts are the following ones:

- Wall: length=*1,70 m*, thickness=*6,00 cm*, height=*0,90 m* (by storey)
- Floor: length=*1,70 m*, width=*1,70 m*, thickness=*0,21 m*
- Footing: length=*2,10 m*, height=*0,60 m*, thickness=*0,10 m*

The total height of the model is 5,10 m.

The walls are loaded in their own plane. The stiffness and the strength in the perpendicular direction are increased by adding some lateral triangular bracing. This system has reduced the risk of failure which might be induced by some parasite transversal motion or a non-symmetric failure of the structural walls. Lateral bracing is such that the two walls carry the entire vertical load. Picture and plan of specimen before test is shown in Figure 3.

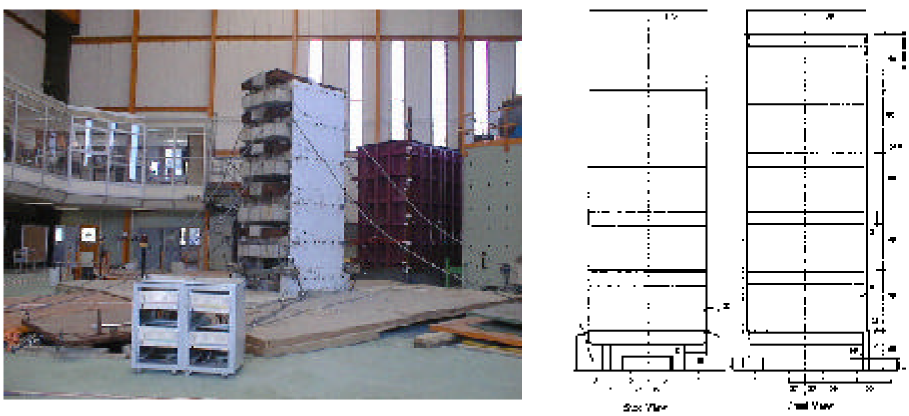


Fig. 3 CAMUS3 specimen in EMSI Sacley laboratory

3.2 Material characteristic

3.2.1 Concrete

The results of the usual compressive tests on 160 mm diameter cylinders show the following average values:

- Strength f_c*=39,6 MPa*,
- Young modulus *E=31139 MPa*,
- Poisson ratio μ*=0,197*.

3.2.2 Steel reinforcement

The specimen CAMUS3 has reinforcement follows the Eurocode 8 provisions. The diameters of the longitudinal reinforcing bars used are *4,5*; *5,0*; *6,0* and *8,0* mm. The *500 MPa* yielding stress was specified. The transversal and confinement steel reinforcement also follows the EC8 provisions:

- Shear strength is ensured by 2 layers of *4,5 mm* diameter horizontal HA steel (one layer on each face) with *175 mm* spacing in the storeys 1 and 2 and *190 mm* in the storeys 3, 4, and 5.
- In plastic hinge region stirrups made of *3,0 mm* diameter bars have been placed on each side of the wall. The stirrups spacing is respectively *20 mm* and *40 mm* for the 1st storey and the 2nd storey. The stirrups are closed with *90°* hoops.
- Tensile yield and failure stresses (f_y / f_u) for the different reinforcing steel bars were:
  F 3,0 mm (*814/849 MPa*); *F 4,5 mm* (*814/849 MPa*); *F 5,0 mm* (*814/849 MPa*); *F 6,0 mm* (*814/849 MPa*) and *F 8,0 mm* (*814/849 MPa*)
  Reinforcement of specimen is shown in Figure 4.

3.3 Mass description

The total mass of the specimen is estimated to be about *36 tons*, about *18 tons* on each wall. Wall mass distribution along the height is shown in Figure 5.

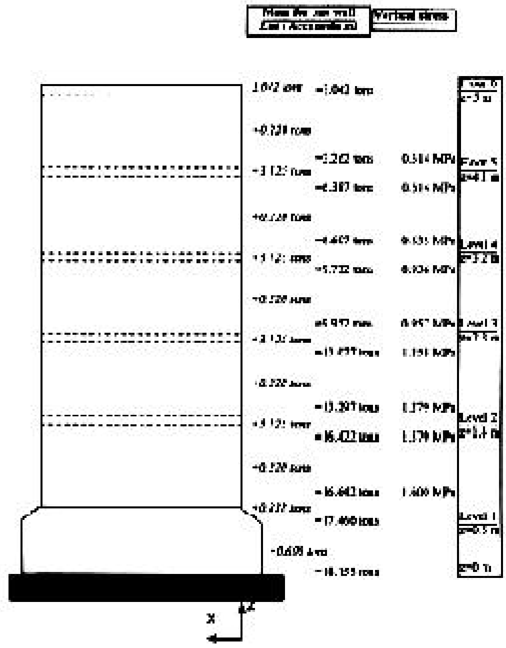


Fig. 5 Wall mass distribution along the height

3.4 Loading program

The synthetic signal NICE S1 representative of the French design acceleration spectra has been used as an input signal. The time scale of the plotted signals is

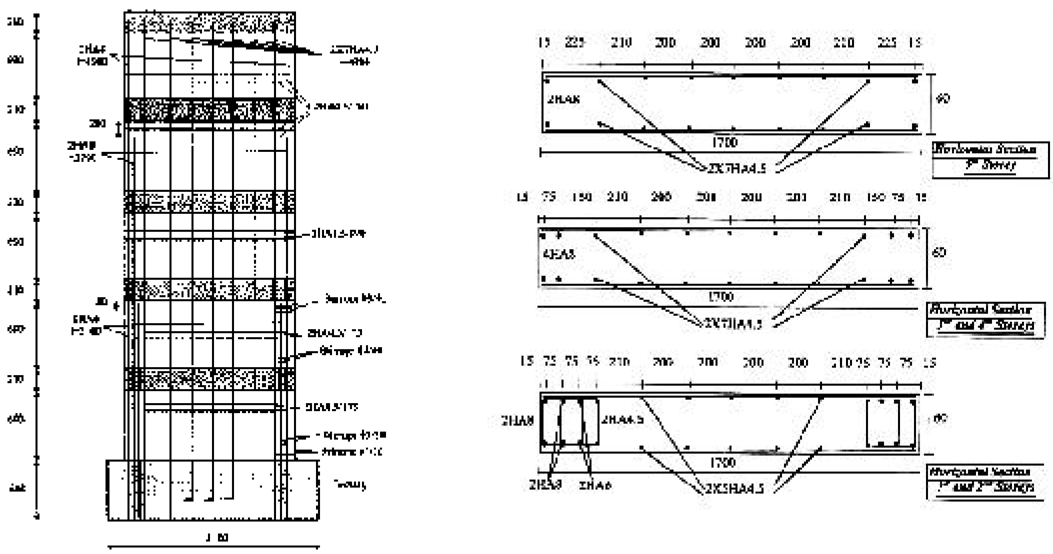
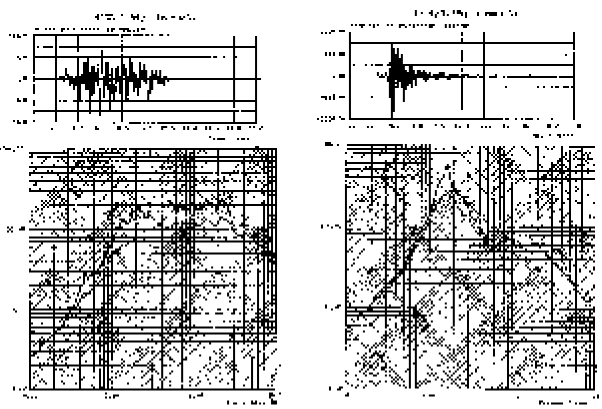


Fig. 4 Reinforcement plan of specimen

already modified in order to take into account the geometrical scale. An intermediate test has been performed with the signal MELENDY RANCH representative of a near-field moderate earthquake. This signal is short but has a high value of acceleration and its frequency content corresponds to the natural frequency of the specimen. Both time-histories and response spectrums for *2%*, *5%* and *10%* are shown on Figure 6. The experiment loading program is given in Table 2.



NICE input *MELENDY RANCH input*

Fig. 6 Loading input time-histories

Table 2 The loading program of the specimen

PHASE NUMBER	LOADING INPUT TIME HISTORY	SCALE FACTOR a/g
1	NICE r6	0,22
2	MELENDY r2	1,35
3	NICE r8	0,64
4	NICE r10	1,02

3.5 Results of experiment

The presentation of main results of the experiment shall be presented, in this chapter, by tables and figures from Final Report [6], without any comments.

3.5.1 Dynamic behaviour of the specimen

Table 3 Dynamic behaviour of specimen

TEST	INITIAL	AFTER NICE 0,22 g	AFTER MELEN. 1,35 g	AFTER NICE 0,64 g	AFTER NICE 1,02 g
NATURAL FREQUENCY (Hz)	6,88	6,44	4,5	4,3	4,49
DAMPING (%) (Natural frequency)	1,94	2,81	3,7	5,4	3,3
DISPLACEMENT RESPONSE (Hz)	6,9	6,0	3,7	3,0	2,3

3.5.2 Global experimental results

3.5.2.1 Base shear - top displacement relations

Table 4 Base shear - top displacement relations

TEST	NICE 0,22 g	MELENDY 1,35 g	NICE 0,64 g	NICE 1,02 g
BASE SHEAR FORCE $(BS)_{max}$ (kN)	48	151	124	140
DISPLACEMENT of FIFTH FLOOR u_{max} (cm)	0,434	2,920	2,750	4,710

3.5.2.2 Displacements of the fifth floor

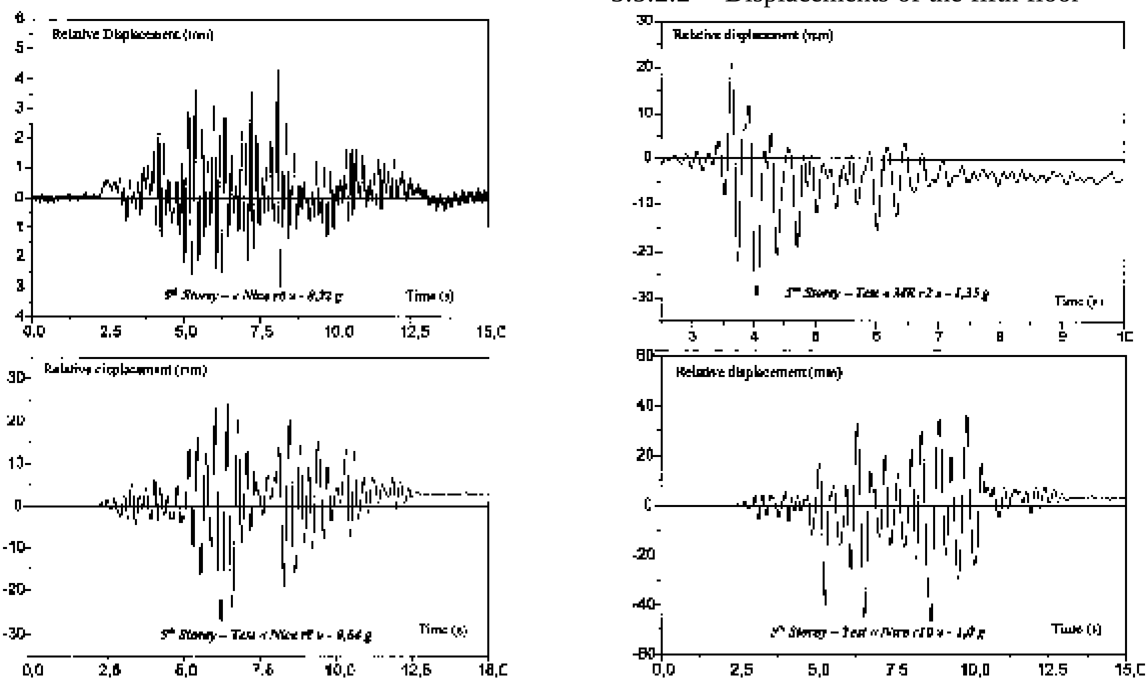


Fig. 7 Displacements response of specimen

3.5.2.3 Base shear - top displacement relations

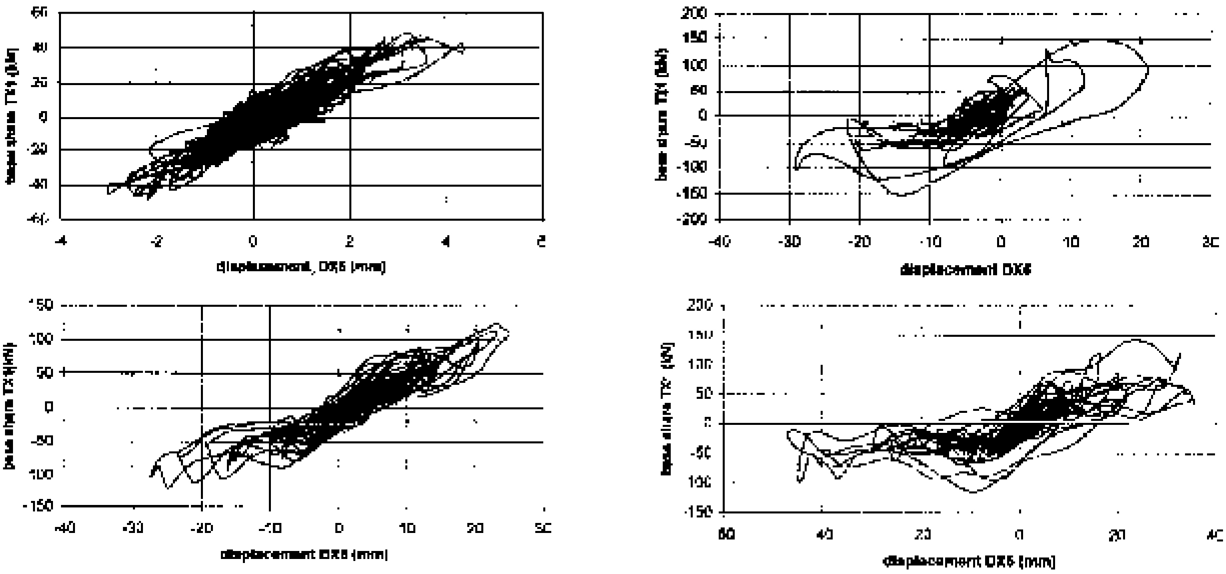


Fig. 8 Base moment - top displacement & base rotation relations

3.5.3 Damage pattern

Description of specimen damage from the Final Report [6]:

Cracks were observed before the tests. These cracks appeared during the assembling of the specimen on the table and particularly during the tightening of the walls on the floors. The main of the cracks were localised at the construction joints. At the top of the specimen we observed on each wall the same crack, in the middle of the walls, in a diagonal direction.

No new cracks (or extension of the existing cracks) appeared until the "MR r2" test (1,35 g). Severe damages were observed on two wall after the test "MR r2". An important crack appeared at the base of each wall along the width. The beginning of spalling was observed at the extremity of the walls. Other cracks were distributed along the 3 first storeys, especially in the diagonal direction. No new cracks (or extension of the existing ones) were observed along the 2 last storeys.

We noticed a slight growing of the cracks after the test "Nice r8" (0,64 g).

Finally, after the last test "Nice r10" (1,02 g), the specimen was heavily damaged at the base, on each wall. On the right wall, spalling at the sought extremity made the steel reinforcement visible. On the left wall, the steel reinforcement was visible at two extremities.

During the disassembling of the specimen, we observed that all the vertical steel reinforcement bars were broken on the left wall, with buckling of the vertical steels. On the right wall, buckling and failure of the steel bars appeared for the vertical steels at one extremity. All other vertical bars were broken. The breaking zone of the steels followed the main cracks at the base.

4. NUMERICAL INTERPRETATION OF SPECIMEN TEST (THE SIMPLIFIED NON-LINEAR ANALYSIS)

In this part of the paper response and damage analysis of specimen's responses for all phases of experiment is given. The analysis consist of the following steps:

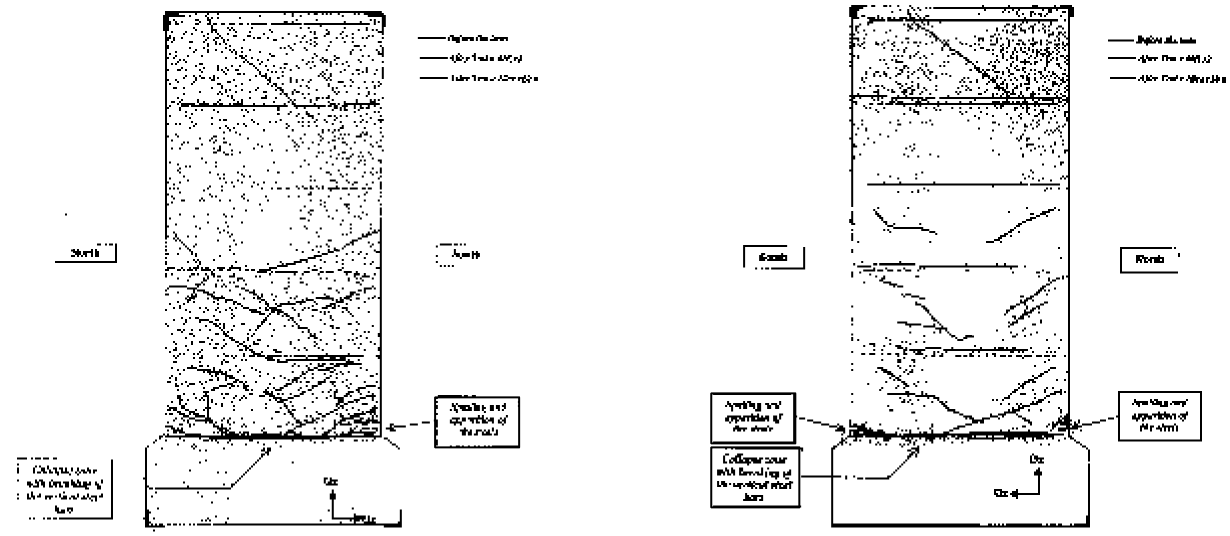
4.1 Creation of the initial SDOF model and conception of calculation approach

SDOF model of specimen will be the model of one wall of specimen. It is necessary to define the following parameters of the model: weight (W), damping (x %), elastic stiffness (K_{EL}), post-elastic stiffness (K_Y) and yield base shear $(BS)_Y$, all for the initial state of specimen:

a) Weight in the base level is known: $W=165$ kN ($s_0=1,6$ MPa);
b) Damping is assumed (R/C wall) with initial value $x_{EL}=2\%$;
c) Initial elastic stiffness $K_{EL}=(GA/1,2h)\ [(1/l+3,33\,(G/E)\,(h/l)^2]=227$ kN/cm;
d) Post-elastic stiffness $K_Y=0,55\,K_{EL}$ (R/C wall post-elastic stiffness based on residual shear stiffness);
e) Dynamic properties $m=W/g=0,168$ kNsec/cm,

$$T=2p\sqrt{m/K_{EL}}=0,171\ sec,\ f=5,85\ Hz;$$

f) Yield base shear $(BS)_Y$ in function of failure mechanism: BENDING or SHEAR capacity must be determined by the following steps:
Cross-section properties:
Concrete C 40/50; $A_c=1020$ cm^2 ($b=6$ cm; $h_0=170$ cm, $h=157,3$ cm)
Reinforced steel $f_y=500$ N/mm^2; $As=As'=2,9$ cm^2 $A_s^{LIN}=2,0$ cm^2/m'
Axial load $N=165$ kN; $s_0=1,6$ MPa.



LEFT WALL RIGHT WALL

Fig. 9 Evolution of the cracking of the specimen's walls

A) BENDING BASE CROSS SECTION CAPACITY

CAPACITY VALUES are calculated using limit state method. Interactive bearing and deformation diaphragms are shown on Figure 10.

Resulting capacity post elastic values, for axial load $N=165\ kN$, are:

- ultimate bending moment $M^U=442\ kNm$
- yield bending moment $M^Y=375\ kNm$
- ultimate rotation $j^U=0{,}021$
- yield rotation $j^Y=0{,}0035$
- rotation ductility $D=5{,}90$

Yield horizontal load distribution:

According to the equivalent static forces method with level high (h_i) and level mass (G_i) (Figure 3) resulting yield base shear is given as:

$$(BS)_Y^{BENDING}=M^Y/3{,}29=375/3{,}29=115\ kN$$

B) SHEAR BASE CROSS SECTION CAPACITY

CAPACITY VALUES defined in [7]:

$v_{cr}=0.17*\sqrt{(f'c)}$ and $g_{cr}=v_{cr}/G$ - cracking shear and deformation,

$v_y=f_y*r$ and $g_y=v_y/G_{cr}$ and $G_{cr}=rnG$ - yielding shear and deformation,

$V_{cr}=v_{cr}*A_{gross}$ and $d_{cr}=H*g_{cr}$ - cracking force and displacement,

$V_y=v_y*A_{gross}$ and $d_y=H*g_y$ - yielding force and displacement,

where: r = ratio of traverse reinforcement; n = ratio between steel and concrete elasticity module; G = shear modulus, A_{gross}=concrete gross area and H=wall height.

According to the presented procedure resulting "shear" yield base shear is:

$$(BS)_Y^{SHEAR}=V_y=324\ kN$$

Accepted initial SDOF yield strength (yield base shear) is $(BS)_Y=115\ kN$, according to the bending failure mechanism.

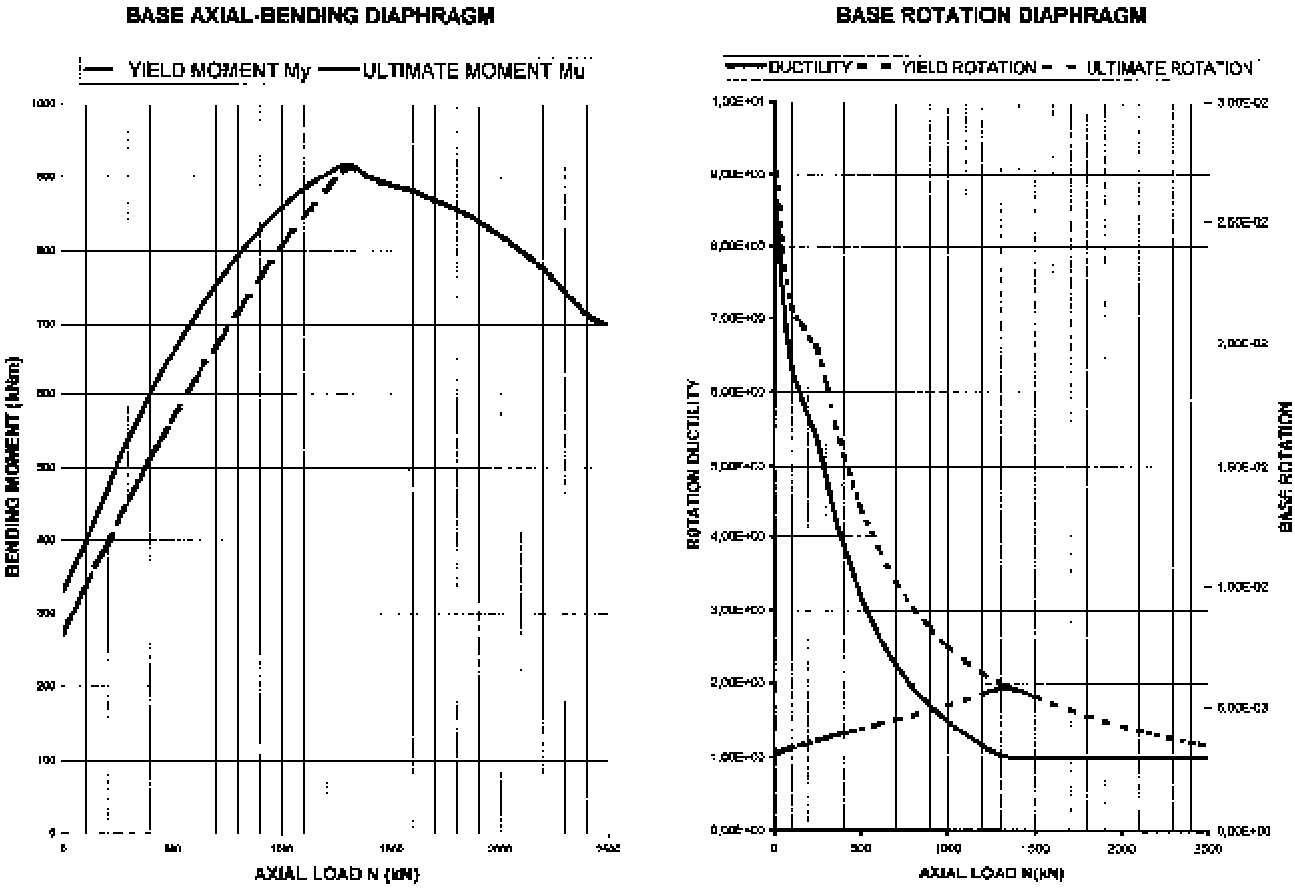


Fig. 10 Interactive bearing and deformation diaphragms of base cross section of wall

4.2 Calculation approach

Calculation approach is based on the assumption that the results of previous phase define initial state of SDOF model for the next phase.

The results of each phase calculation are:

- Time-history displacements response of structure (maximum value u^{max}) and FFT analysis in frequency domain of that response.
- Base shear (*BS*) - displacement hysteresis during the all response time.
- Number of yield excursions reached during the response (N_Y).
- Cumulative energy balance with energy dissipated by hysteresis, during the earthquake (E_H).
- Displacement ductility demand ($D=u^{max}/u^y$).

Based on the results of SDOF model's response, two steps of its analysis are necessary:

1. To calculate damage of structure by damage ratio (DR) after each phase (according to Eq. (1)).
2. To calculate new parameters of SDOF model for next phase:
   - Elastic stiffness $K'_{EL.}=(BS)_{max}/u^{max}$ from previous phase (residual secant stiffness);
   - Residual yield base shear (according to Eq. (2));
   - Increase of damping coefficient (according to Eq. (3)).

In such a manner the modified initial SDOF model presents structure at the beginning of next phase.

4.3 Simplified non-linear analysis (Results of provided calculations)

All calculations are provided by program NONLIN [8], step by step time-history numerical integration. The results of provided analysis (time-history of top displacements with FFT analysis in frequency domain, base shear-displacements hysteresis response, yield excursions and cumulative energy transformation) are given, for all four phases of the experiment.

4.3.1 Phase 1.

Input load: NICE *0,22 g*, Figure 11.

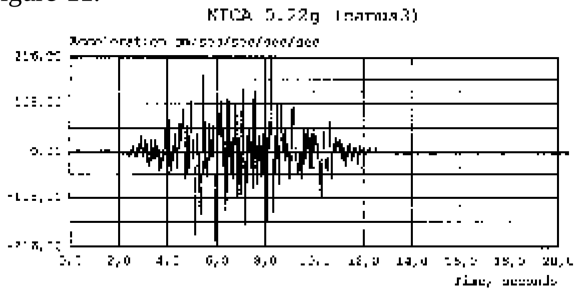


Fig. 11 Input load NICE 0,22 g

Table 5 SDOF model parameters, phase 1

Weight W (kN)	Stiffness K_{EL} (kN/cm)	Strain hard. stiffnes K^{Y} (kN/cm)	Yield strength S^{Y} (kN)	Yield displac. u^{y} (cm)	Damping î (%)	Fundam. period T (sec)	Cyclic frequency f (Hz)
165	227	125	115	0,507	2	0,171	5,85

Structure response, phase 1:

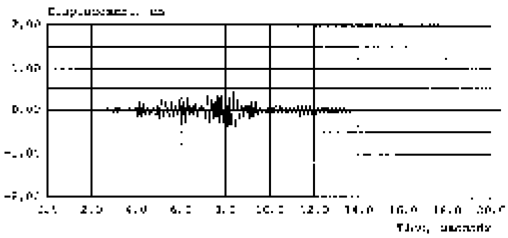


Fig. 12 Response displacement time-history

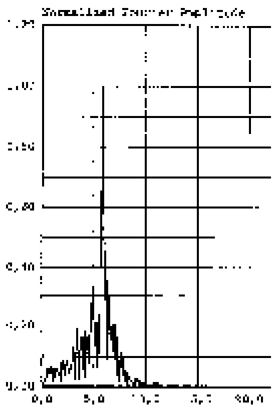


Fig. 13 FFT analysis

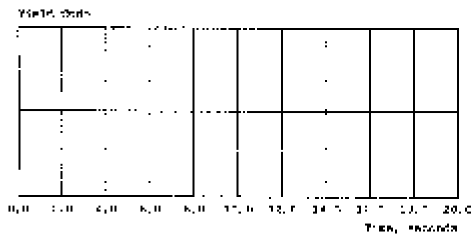


Fig. 14 Time-history of yield excursions (N_Y)

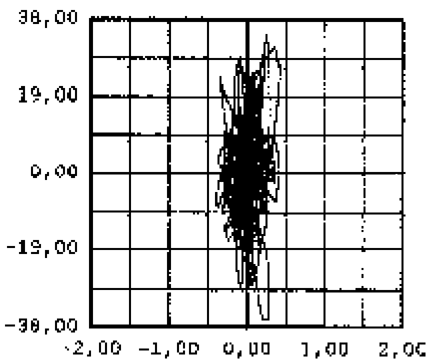


Fig. 15 Base shear (BS) - displacement hysteresis

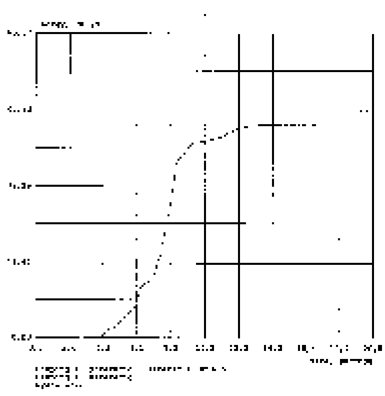


Fig. 16 Cumulative energy balance

Table 6 Summary of response values, phase 1

u^{max} (cm)	Displ. freq. (Hz)	u^{y} (cm)	BS_{MAX} (kN)	K^{SEC} (kN/cm)	E_{TOT} (kNcm)	E_H (%)
0,428	6,0	0,507	36,34	227	47	0

Table 7 Summary of damage analysis, phase 1

D	$\frac{K_{EL}}{K^{SEC}}$	N_Y	E_H (kNcm)	DR Eq. (1)	S_Y^{RES} Eq. (2)	$î^{RES}$ Eq. (3)
<1	1	0	0	0,00	115	2,00

4.3.2 Phase 2.

Input load: MELENDY RENCH *1,35 g*, Figure 17.

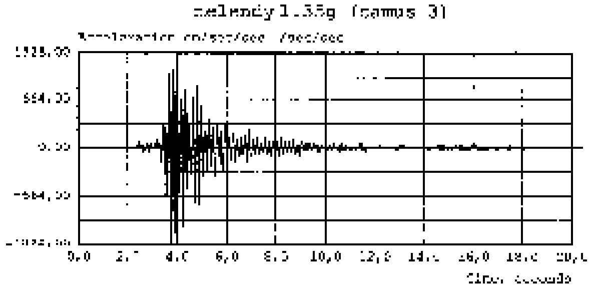


Fig. 17 Input load MELENDY RENCH 1,35 g

Table 8 SDOF model parameters, phase 2

Weight W (kN)	Stiffness K_{EL} (kN/cm)	Strain hard. stiffness K^{Y} (kN/cm)	Yield strength S^{Y} (kN)	Yield displac. u^{y} (cm)	Damping î (%)	Fundam. period T (sec)	Cyclic frequency f (Hz)
165	227	125	115	0,507	2	0,171	5,85

Structure response, phase 2:

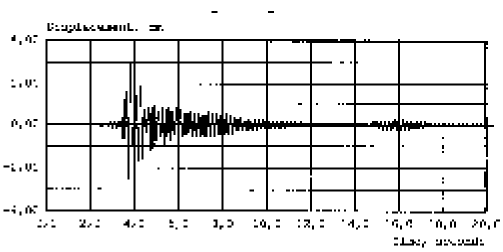


Fig. 18 Response displacement time-history

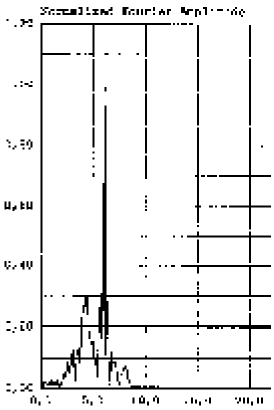


Fig. 19 FFT analysis

Fig. 20 Time-history of yield excursions (N_Y)

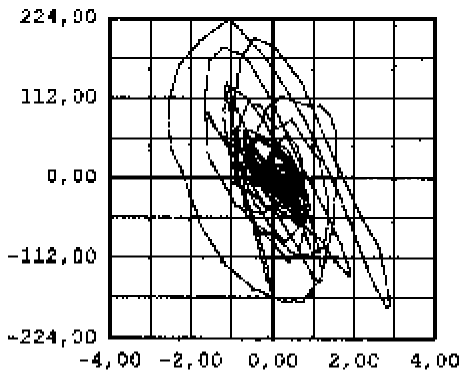


Fig. 21 Base shear (BS) - displacement hysteresis

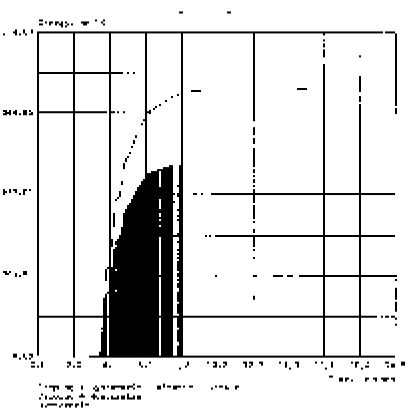


Fig. 22 Cumulative energy balance

Table 9 Summary of response values, phase 2

u^{max} (cm)	Displ. freq. (Hz)	u^{y} (cm)	BS_{MAX} (kN)	K^{SEC} kN/cm)	E_{TOT} (kNcm)	E_H (%)
2,911	5,85	0,507	176	61	2645	71

Table 10 Summary of damage analysis, phase 2

D	$\frac{K_{EL}}{K^{SEC}}$	N_Y	E_H (kNcm)	DR Eq. (1)	S_Y^{RES} Eq. (2)	$î^{RES}$ Eq. (3)
5,75	3,75	54	1878	0,599	73	3,16

4.3.3 Phase 3.

Input load: NICE *0,64 g*, Figure 23.

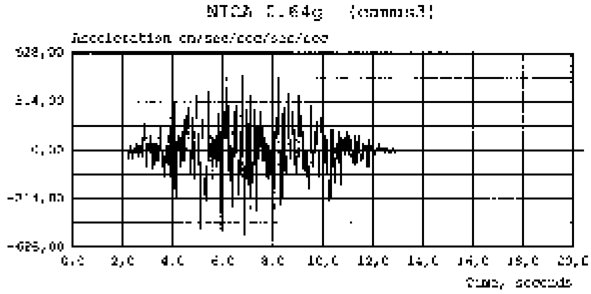


Fig. 23 Input load NICE 0,64 g

Table 11 SDOF model parameters, phase 3

Weight $W(kN)$	Stiffness K_{EL} (kN/cm)	Strain hard. stiffness K^{Y} (kN/cm)	Yield strength S^{Y} (kN)	Yield displac. u^{y} (cm)	Damping î (%)	Fundam. period T (sec)	Cyclic frequency f (Hz)
165	61	33	73	1,202	3,16	0,331	3,021

Structure response, phase 3:

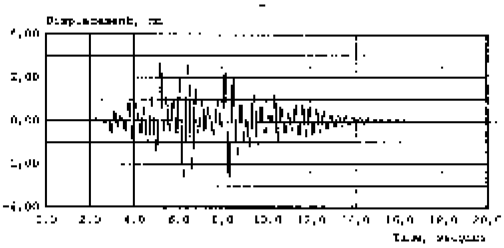


Fig. 24 Response displacement time-history

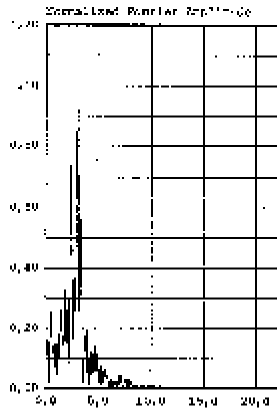


Fig. 25 FFT analysis

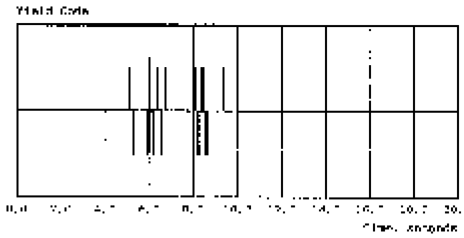


Fig. 26 Time-history of yield excursions (N_Y)

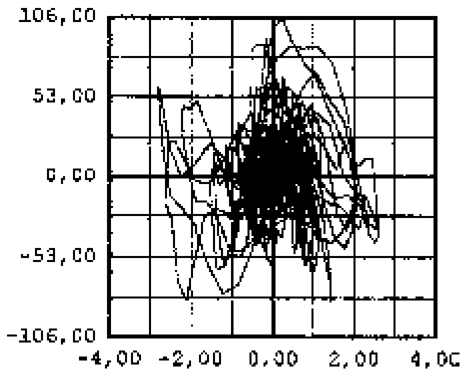


Fig. 27 Base shear (BS) - displacement hysteresis

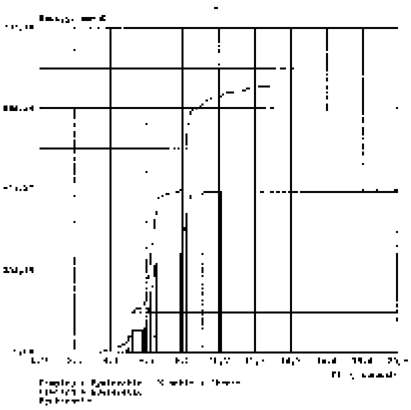


Fig. 28 Cumulative energy balance

Table 12 Summary of response values, phase 3

u^{max} (cm)	Displ. freq. (Hz)	u^{y} (cm)	BS_{MAX} (kN)	K^{SEC} kN/cm)	E_{TOT} (kNcm)	E_H (%)
2,806	3,00	1,202	106	38	1064	60

Table 13 Summary of damage analysis, phase 3

D	$\frac{K_{EL}}{K^{SEC}}$	N_Y	E_H (kNcm)	DR Eq. (1)	S_Y^{RES} Eq. (2)	$î^{RES}$ Eq. (3)
2,34	1,61	15	638	0,26	63	3,67

4.3.4 Phase 4.

Input load: NICE *1,02 g*, Figure 29.

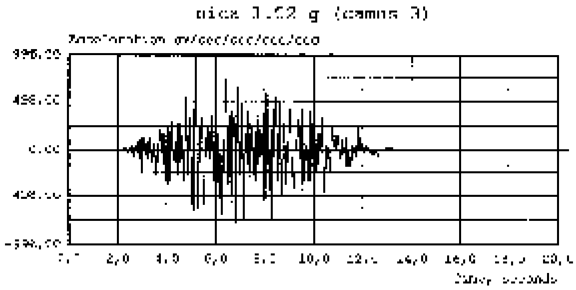


Fig. 29 Input load NICE 1,02 g

Table 14 SDOF model parameters, phase 4

Weight W(Kn)	*Stiffness K_{EL} (kN/cm)*	*Strain hard. stiffness K^Y (kN/cm)*	*Yield strength S^Y (kN)*	*Yield displac. u^y (cm)*	*Damping î (%)*	*Fundam. period T (sec)*	*Cyclic frequenc f (Hz)*
165	*38*	*21*	*63*	*1,663*	*3,67*	*0,420*	*2,381*

Structure response, phase 4:

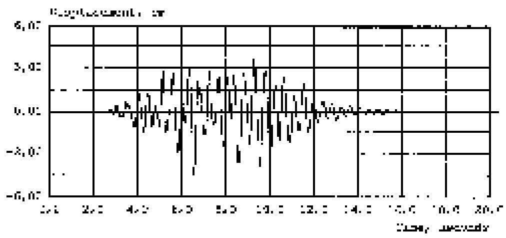


Fig. 30 Response displacement time-history

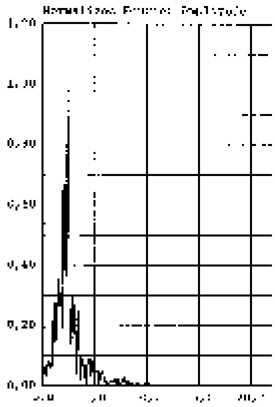


Fig. 31 FFT analysis

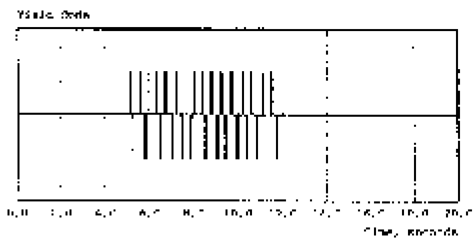


Fig. 32 Time-history of yield excursions (N_Y)

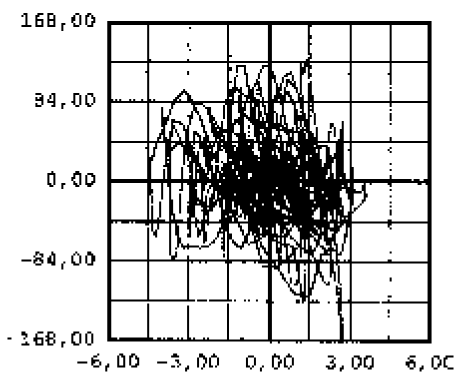


Fig. 33 Base shear (BS) - displacement hysteresis

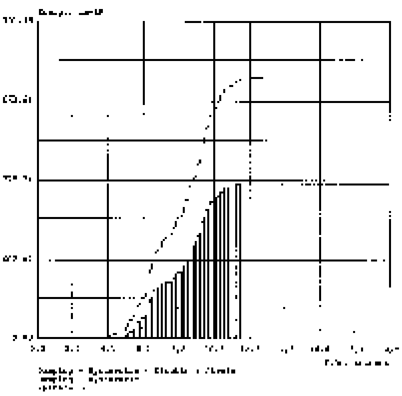


Fig. 34 Cumulative energy balance

Table 15 Summary of response values, phase 4

u^{max} (cm)	*Displ. freq. (Hz)*	*u^y (cm)*	*BS_{MAX} (kN)*	*K^{SEC} (kN/cm)*	*E_{TOT} (kNcm)*	*E_H (%)*
4,437	*2,40*	*1,663*	*126*	*28*	*2260*	*60*

Table 16 Summary of damage analysis, phase 4

D	*$\frac{K_{EL}}{K^{SEC}}$*	*N_Y*	*E_H (kNcm)*	*DR Eq. (1)*	*S_Y^{RES} Eq. (2)*	*$î^{RES}$ Eq. (3)*
2,668	*1,33*	*28*	*1356*	*0,337*	*51*	*4,51*

5. COMPARISON OF RESULTS AND DISCUSSION

5.1 Response parameters comparison

Response parameters values given by SDOF model analysis which is possible to be compared with test results are: the maximum response displacement (u^{max}) with its frequency and the base shear (*BS*) in each phase, and changing values of damping coefficient (x), during the test, after each phase. Comparison is given by the following tables and figures.

Table 17 Top displacements [u (cm)] comparison

PHASE OF TEST	EXPERIMENT	CALCULATION	RELATION (Cal./Exp.)
NICE 0,22g	0,434	0,438	1,01
MELENDY 1,35g	2,920	2,911	1,00
NICE 0,64g	2,750	2,806	1,02
NICE 1,02g	4,710	4,437	0,94

Table 18 Base shear [BS (kN)] comparison

PHASE OF TEST	EXPERIMENT	CALCULATION	RELATION (Cal./Exp.)
NICE 0,22g	48	36	0,75
MELENDY 1,35g	151	176	1,16
NICE 0,64g	124	106	0,85
NICE 1,02g	140	126	0,90

Table 19 Frequency of displacements response change comparison

PHASE OF TEST	EXPERIMENT	CALCULATION	RELATION (Cal./Exp.)
INITIAL	6,90	6,00	0,87
AFTER NICE 0,22g	6,00	6,00	1,00
AFTER MELEN. 1,35g	3,70	3,00	0,81
AFTER NICE 0,64g	3,00	2,33	0,78
AFTER NICE 1,02g	2,30	2,00	0,87

Table 20 Damping of specimen and SDOF model change comparison

PHASE OF TEST	EXPERIMENT	CALCULATION	RELATION (Cal./Exp.)
INITIAL	1,94	2,00	1,03
AFTER NICE 0,22g	2,81	2,00	0,71
AFTER MELEN. 1,35g	3,70	3,16	0,85
AFTER NICE 0,64g	5,40	3,67	0,68
AFTER NICE 1,02g	3,30	4,51	1,37

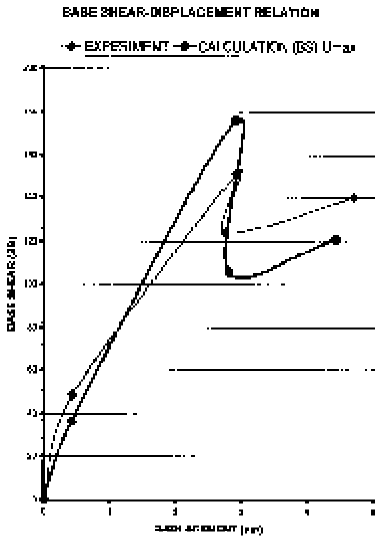


Fig. 35 Base shear-displacement envelope

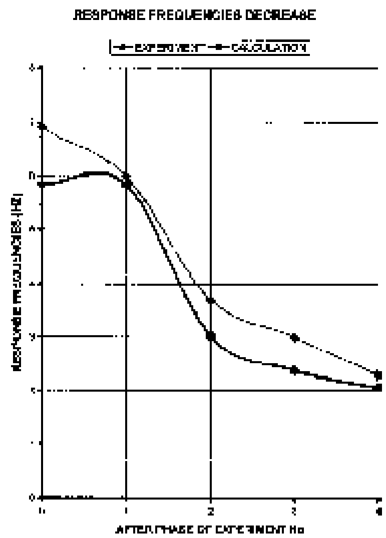


Fig. 36 Response frequencies envelope

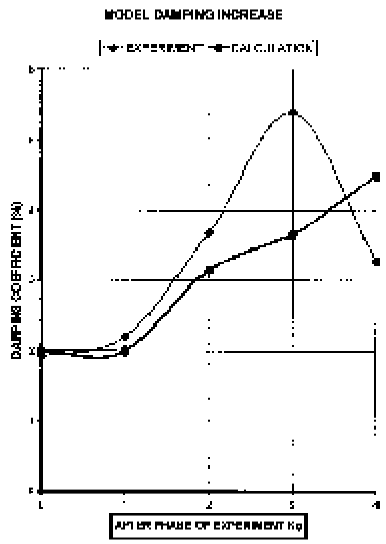


Fig. 37 Damping coefficient envelope

It is significant a very low level of differences between directly measured and calculated maximum displacements in all four phases. That fact approved all presumptions of calculation approach and basic hypothesis for elastic and post-elastic behaviour of analyzed structure.

Differences between "measured" and calculated values of base shear can be caused by procedure of its determination. "Measured" value is in function of measured acceleration, while calculated value is produced by summing inertial, damping and spring component of base shear.

Envelopes of base shear-displacement relation given by measured and calculated values show the same differences in stiffness caused by differences in base shear values, but the similarity in envelope shapes shows an acceptable interpretation of behaviour and mechanism given by calculation.

Differences between displacements response frequencies caused evidently due to the differences in stiffness are influenced by the space stiffness of specimen. Floor structures evidently increase global stiffness of specimen structure, which was ignored in SDOF model.

Huge differences in damping coefficient after phase 3 and phase 4 are caused by non-logic change in "measured" values of damping. Values "must" increase after each phase in which post-elastic deformations happened. Strong increasing after phase 3 (in which damage analysis shows low increase of damage) and equal strong decreasing after phase 4 (in which damage analysis shows increase of damage) is confused.

5.2 Damage analysis

Let's see the basic parameters of SDOF model characterizing specimen at the beginning of the test and the same ones at the end of the test.

Basic global parameters of structure: stiffness, damping and its increase, and yield strength with its degradations, can show the state of structural damage. Using relations defined with Eqs. (1), (2) and (3) for each phase of the test, decrease of yield strength increase of damping and damage ratio were calculated. At the end of test cumulative value of damage ratio was:

$$DR_{CUMULATIVE}=0{,}599(1+0{,}26+(0{,}26\cdot 0{,}337))=0{,}81$$

and the global relations of "damage" parameter is shown in Table 23 and Figures 38, 39, 40.

Let's remind, in Chapter 3.5.3, damage descriptions:

"No new cracks (or extension of the existing cracks) appeared until the "MR r2" test (1,35 g)"

"Severe damages were observed on two walls after the test "MR r2""

"We noticed a slight growing of the cracks after the test "Nice r8" (0,64 g)"

"Finally, after the last test "Nice r10" (1,02 g), the specimen was heavily damaged at the base, on each wall."

Compare it with cumulative values of damage ratio after each phase:

After phase 1. Nice *0,22 g*: *DR=0,00; S=0*
No damage

After phase 2. Mel *1,35 g*: *DR=0,60; S=4*
Severe damage, repairable

After phase 3. Nice *0,64 g*: *DR=0,75*; *S=4*
Severe damage, repairable

After phase 4. Nice *1,02 g*: *DR=0, 81; S=5*
Heavy damage, repairable

It is possible to declare that the proposed damage model *DR*, Eq. (1), and the residual values of seismic resistance S_y, Eq. (2), and damping x, Eq. (3), quite correctly define the level of damage and residual state of structure after earthquake.

Residual yield resistance was *0,443* of initial and damping coefficient was *2,255* of initial. These parameters define state of specimen at the end of test.

Table 21 SDOF model parameters at the beginning of test

Weight W(kN)	Stiffness K_{EL} (kN/cm)	Strain hard. stiffness K^Y (kN/cm)	Yield strength S^Y (kN)	Yield displac. u^y (cm)	Damping î (%)	Fundam. period T (sec)	Cyclic frequency f (Hz)
165	227	125	115	0,507	2	0,171	5,85

Table 22 SDOF model parameters at the end of test

Weight W(kN)	Stiffness K_{EL} (Kn/cm)	Strain hard. stiffness K^Y (kN/cm)	Yield strength S^Y (kN)	Yield displac. u^y (cm)	Damping î (%)	Fundam. period T (sec)	Cyclic frequency f (Hz)
165	28	16	51	1,796	4,51	0,480	2,07

Table 23 Damage analysis at the end of test

RESIDIUAL STIFFNESS $K^{RES} / K^{INIT.}$	RESIDIUAL YIELD STRENGTH S_Y^{RES} / S_Y^{INIT}	RESIDIUAL DAMPING $\hat{\imath}^{RES} / \hat{\imath}^{INIT}$	CUMULATIVE DAMAGE RATIO $DR_{CUMULATIVE}$	Code damage level (S) (1° to 6°)
28 / 227 = 0,123	51 / 115 = 0,443	4,51 / 2 = 2,255	0,81	5°- HEAVY

Concentration of damage was at the base on each wall of specimen, which means developing of plastic hinge caused by bending moment. That fact makes initial analysis of failure mechanism correct.

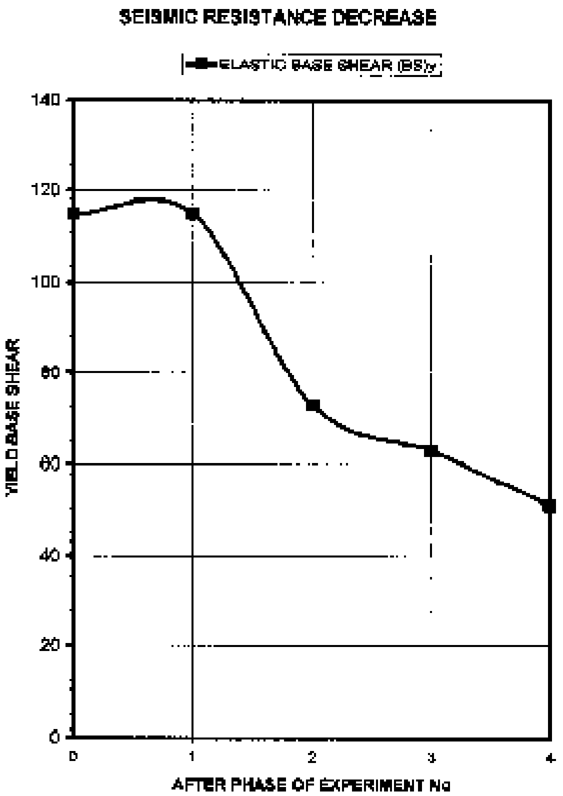


Fig. 38 Nominal seismic resistance

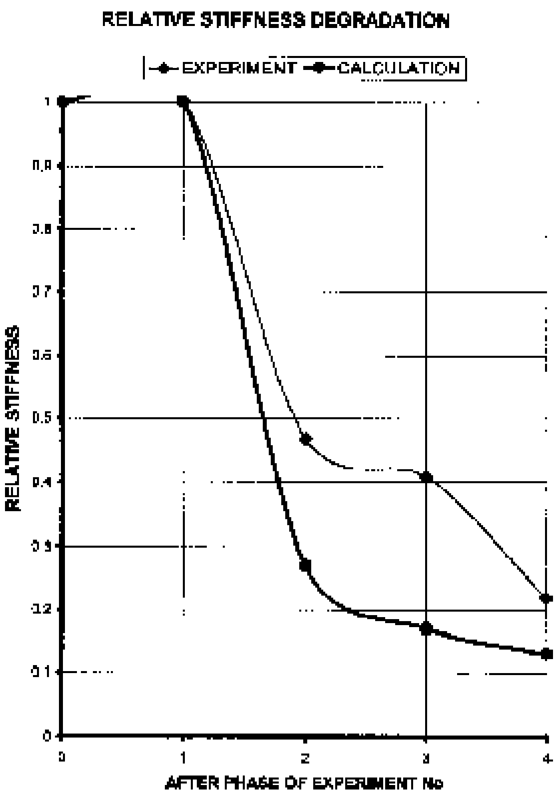


Fig. 39 Relative stiffness degradation

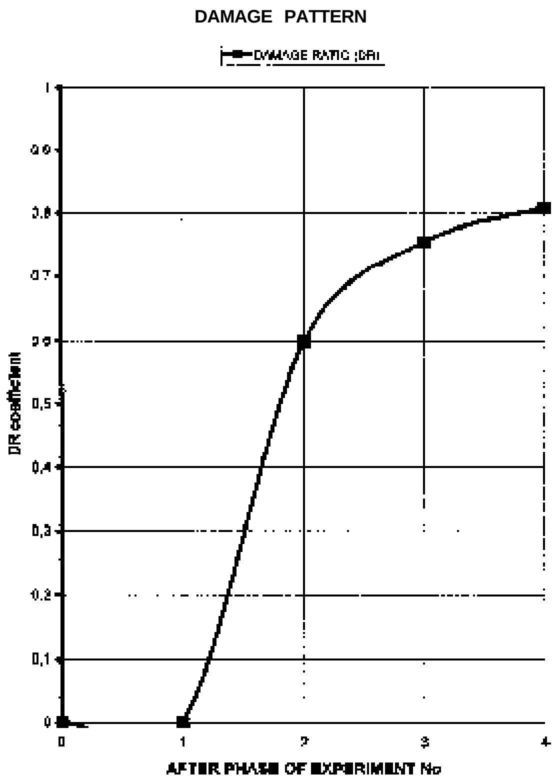


Fig. 40 Damage ratio (DR) of specimen

6. CONCLUSIONS

Seismic response of regular structures (symmetric plans and constant vertical stiffness) can be well interpreted by using SDOF system as a mathematical model of the structures. Response parameters of structure as ductility, stiffness change, energy balance and a number of plastic excursions can describe a real level of structural damage.

It was confirmed by comparison of all these values measured during CAMUS3 experiment and the calculated ones.

Proposed method of calculation is quite acceptable for regular structures earthquake analysis in engineering practice.

Level of structural damage (damage ratio *DR*) can be described by combination of the following calculated structure response parameters: displacement ductility (*D*), stiffness degradation (*DK*), number of yield excursions (N_Y) and hysteresis energy (E_H) per mass unit, by the formula:

$$DR = \frac{1}{30}\left[D + DK + \sqrt[3]{\left(N_Y E_H / W\right)}\right]$$

It is possible to declare decreased residual seismic resistance and increased residual damping coefficient of structure after earthquake using the following two formulas:

$$S_Y^{RESIDUAL} = S_Y^{INITIAL} \cdot \sqrt{(1 - DR)}$$

$$x^{RESIDUAL} = \frac{x^{INITIAL}}{\sqrt{(1 - DR)}}$$

7. REFERENCES

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MODEL SEIZMIÈKE OSJETLJIVOSTI ZA PRAVILNE KONSTRUKCIJE

SAŽETAK

U radu se analizira model koeficijenta oštećenja pravilnih konstrukcija pri potresu. Pri tome se usvajaju sljedeæe pretpostavke:

- *Odaziv pravilne konstrukcije (simetrièni tlocrt i konstantna vertikalna krutost) pri djelovanju potresa može se korektno odrediti koristeæi sustav s jednim stupnjem slobode (SDOF) kao matematièki model konstrukcije.*
- *Parametri odaziva konstrukcija kao što su duktilitet, promjena krutosti, kumulativna analiza oblika energije tijekom potresa i broja ciklusa u kojima dolazi do prekoraèenja granice elastiènosti , mogu opisati stvarno dosegnuti nivo ošteæenja konstrukcije.*

Na temelju nekih prihvaæenih modela koeficijenta ošteæenja, poznatih i prihvaæenih, predlaže se prilagođen moguænostima proraèuna, originalni izraz za izraèun koeficijenta ošteæenja konstrukcije nakon potresa.

Navedene pretpostavke i toènost predloženog modela za analizu ošteæenja vrednovana je usporedbom rezultata proraèuna provedenih u skladu s predloženim modelom s rezultatima provedenog eksperimenta Camus3 na vibroplatformi u EMSI u Sacley Francuska.

Kljuène rijeèi: *model seizmièke osjetljivosti, pravilne konstrukcije, određivanje seizmièke otpornosti.*