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STRUCTURAL CRITERIA TO MEASURE THE BENEFIT OF RETROFIT MEASURES FOR USE IN INTERDISCIPLINARY EFFICIENCY STUDIES

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SUMMARY

Within this study the seismic performance of reinforced concrete frame buildings subjected to cyclic bending has been assessed. Outlooking to a general methodology of the author to assess post-damage repair versus preventive retrofit costs, damage in following cases has been considered: unretrofitted building; retrofit of undamaged and previously damaged building. The innovative part lies in the stress-strain based approach applied to models of building size. Such an analysis allows not only description of failure mode and determination of limit states eventually reached by the building, but also the specific determination of the number and position of structural members suffering different types of damage. This kind of output can build the input for interdisciplinary studies, like the economic study mentioned above. The motivation for the study has been to develop measurable criteria to assess the benefit of retrofit measures on buildings in historic context, of use for cost-benefit analysises, from structural point of view. The stress-strain based approach permits assessment of predicted damage according to European Macroseismic Scale. To determine the impacts of strong ground motion on a certain building types, several kinds of analysis have been applied, first to simple models at laboratory scale, then to progressively complicated models, from regular frame structures, to real buildings: static pushover curve; displacement based dynamic time-history; stress-strain approach based on the dynamic time-history. Several computer tools, including fibre based finite elements, spread sheet and database programs have been used. Criteria to survey characteristics of building stock to construct such models are also presented. The general validity of the prediction has been studied extensively, comparing the computed damages types with real ones and proved to be close to reality. For this purpose a matrix of damages suffered by typical buildings, developed by the author, has been used. The pathology described is supported by sketches.

INTRODUCTION

Retrofitting existing buildings is an important method to raise the safety of built substance in regions of earthquake peril. Only few studies on the costs of retrofit measures have been completed until today. Of relevance for the topic of this paper are those of ATC[1] and of FEMA[2]. None of them results in a "family of curves", which is the purpose of the method to be presented. In order to determine the

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economic efficiency both the retrofit costs before and the retrofit costs after an earthquake will be regarded. Aim of the study is to show an algorithm therefore. The assumptions made are clearly shown and thus the way for employing optimal models into this algorithm is open.

MODELS AND RETROFIT METHODS

The computer program employed in the current work is SeismoStruct[3], a free fibre-modelling Finite Element program for seismic analysis of framed structures. This package is capable of predicting the large displacement behaviour of space frames under static or dynamic loading, taking into account both local (beam-column effect) and global (large displacements/rotations effects) geometric nonlinearities as well as material inelasticity. Its spread along the member length and across the section area is explicitly represented through the employment of a fibre modelling approach, implicit in the formulation of the inelastic beam-column frame elements employed in the analyses.

Simplified structures

Model “Gregor” consists of a regular structure, 22m high, 20m long and 15m wide. It has six floors, four bays and three frames, identical in both directions: 3m high and 4.5m span, with 50cm square columns and 50cm high beams. The column reinforcement consists of Φ14-16mm Re-bars spaced 25-30cm and Φ6-8mm stirrups spaced 25-35cm, both out of smooth steel, the last ones not required to be modelled in the program employed. Concrete had an average resistance of 150 daN/cm^2. For beam reinforcement the assumption of typical reinforcement for beams in a building designed today was made. The first measure considered regards the addition of side walls to columns (layout in fig. 1, details in fig. 4 and table 1). On the long side walls are added left and right to the middle column. On the short side one side wall is added to each of the two columns in the middle within the same frame. This measure will assure not only increase of strength but also ductility of the building. The second retrofit measure considered was the jacketing with steel of half of column height (layout in fig. 1, details in fig. 5). The advantage of this method lays in almost no increase of the section. Other two measures are the addition of a complete structural wall within a vertical row of frames (fig. 2) and the alternative addition of steel braces within the same frames. In case of addition of structural wall the new panel is connected to the frame respectively the foundation on all four sides, usually using chemical fixation with epoxy resins of the panel Re-bars into previously drilled holes in the reinforced concrete frame. An enlargement of the foundation is also required (fig. 6). The addition of a reinforced concrete structural wall within the whole frame leads to a strength increase of the building. Braces in “V” shaped layout are to be fixed on a second, metallic, frame, within the concrete frame, as the quality of concrete in existing building can be considered not very reliable for direct fixing in nodes (fig. 7). The second model considered, called Model "Özzi" is similar to the one just described (fig. 3). The building’s main part has five storeys only, five bays and also three frames. For frames with windows a different, smaller span of 3.5m was adopted.

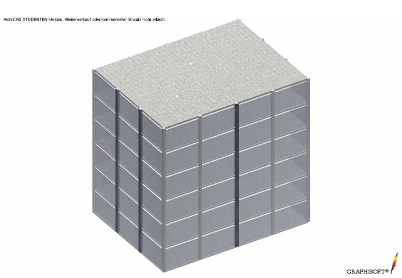


Figure 1: Layout of columns in model “Gregor” on which retrofit with sidewalls or metal jacket was applied

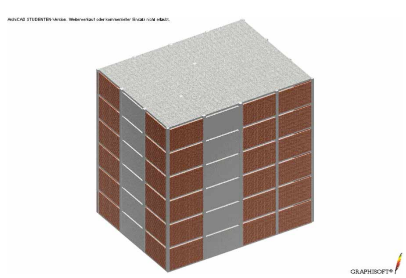


Figure 2: Layout of frames in model “Gregor” in which retrofit with structural walls or diagonal steel braces was applied

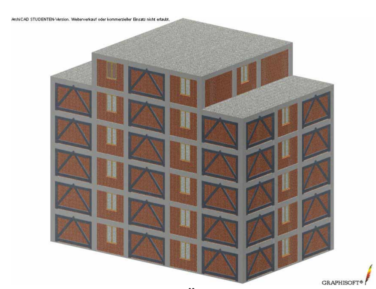


Figure 3: Model “Özzi” with layout of proposed retrofit measures in the 5th and “strongest” variant

Table 1: Device directory for retrofit with side walls (after Bostenaru[4])

No.	Work	Unit	work time/ unit	price/ unit (€)	No. units	price/ time (€)	total price (€)
1	Scaffolding	m²		8	2.88		23.04
2	Screening	piece	2h/12		1	36	6
3	Setting up and removing drop tub	piece	4h/12		1	36	12
4	Partial breaking of masonry wall	m³	5.6h		1.05	36	211.68
5	Unloading of column through bolts	piece	3h		1	36	108
6	Cleaning up masonry	m³		19	1.05		28.5
7	Reinforcing	kg	60kg/h		86.4	36	51.84
	Steel	kg		0.5	86.4		43.2
8	Reinforcement anchoring in existing RC frame	piece	5h		1	36	180
	Anchor elements	piece		2.83	30		84.9
9	Setting up formwork	piece	2h		1	36	72
	Wooden formwork	m²		8.5	6		51/4
	Formwork support (scantling)	m		0.55	22		12.1/4
10	Casting concrete	m³	1h		1	36	36
	Concrete	m³		140	0.72		100.8
11	Removing formwork	piece	2h		1	36	72
	PRICE	wall element					1929.37

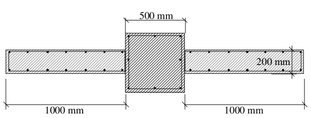
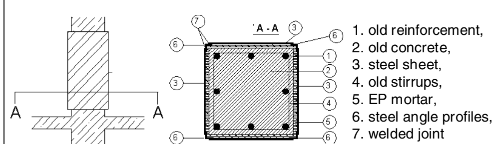
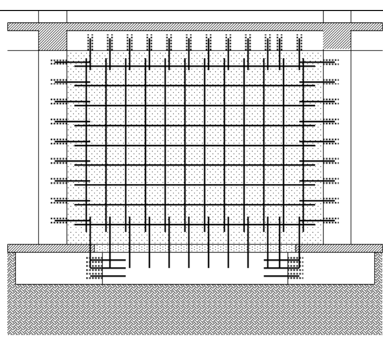


Figure 4: Sidewall on chosen columns (after Bostenaru[])



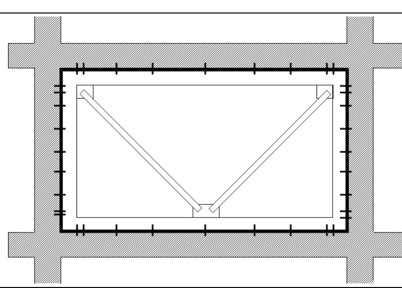
No.	Work	total price (€)
1	Roughening concrete surface	72
2	Roughening steel surface	72
3	Mounting steel on the RC column	1266
4	Filling between steel and concrete	122
	PRICE	1532

Figure 5: Steel jacketing of columns (after Bostenaru[4])



No.	Work	total price (€)
1	Scaffolding	120
2	Screening	12
3	Building up and removing drop tub	24
4	Breaking up the masonry wall	237.6
5	Breaking through the slab	180
6	Cleaning up the brocken masonry	78.8
7	Reinforcement	380.16
8	Anchoring the reinforcement in existing RC frame	529.8
9	Setting up formwork	251.7
10	Casting concrete	342
11	Removing formwork	72
	Foundation	
	PRICE	2228.06

Figure 6: Structural walls within chosen frames (after Bostenaru[4])



No.	Work	total price (€)
1	Scaffolding	120
2	Screening	12
3	Setting up and removing drop tub	24
4	Breaking up the masonry wall	836.08
5	Cleaning up masonry	78.85
6	Steel frame and diagonals mounting into RC frame	2328.4
	PRICE	3399.33

Figure 7: Steel diagonals within chosen frames (after Bostenaru[4])

Models of buildings

The retrofit measures described are to be applied at facade elements. This is a favourable situation, as in building survey facade elements are easier to record and they can also be used to estimate vulnerability in a simplified way (Glaister[5]). Thus the conclusions drawn out by studying these models could be potentially applied at large urban scale. Before using the results in a scenario at larger urban scale, the applicability for real buildings was checked.

Model "Interbelic"

This structural model of a real building with ground floor, basement and five upper floors has a relatively regular structure in height, by having only the basement different from the upper floors, but a highly irregular structure in plan, due to the distribution of columns but mainly to those of beams. The beams are not directly supported by columns in most of the cases, but by other beams and there are consoles at two of the four facades (fig. 7). The same reinforcement details used for the model buildings with regular shape described previously were taken. A proposed retrofit scheme with structural walls is also available. However, this does not fit in any of the schemes previously described (fig. 8). For this building more methods have been combined into a retrofit strategy. They include addition of structural walls and jacketing of columns. Structural walls are added on both sides of the middle bays and on the corresponding frames on the short sides of the building. On the long side four structural walls are added symmetrically to the centre of the building. These do not fill a frame as the existence of windows has been taken into consideration. The structural walls have much wider sections (25-30cm) as usually for such (15cm in new buildings, 20cm in the regular structure models here). Finally, two of the middle most solicited columns have been jacketed. The effect of the measures is a reduction of irregularity in plane. On the other side, the solution proposed presents the disadvantages described of increasing the section of structural members, altering the façade. It was tried though, that the intervention does not alter the quality of space. The structural walls present barbell sections, which could not be modelled for the analysis. For the analysis 10 bars of 28mm in each jacket were considered. For structural walls the same reinforcement bars were used. The retrofit could be well modelled in the software package used. For the column jacketing a special defined section was available in the software, which takes into account the bond between new concrete in the jacket and the old concrete in the original member. A fine FEM grid could be provided only with two elements for each member, due to model complexity and the already high number of nodes.

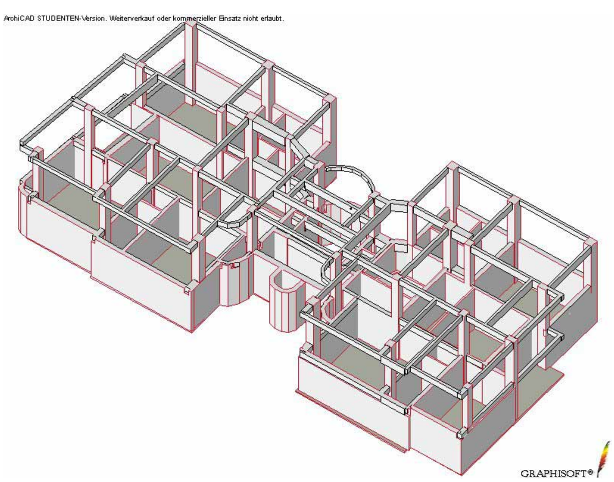


Figure 8: Axonmetrie of current floor and structure of "Interbelic" (from Bostenaru[4])

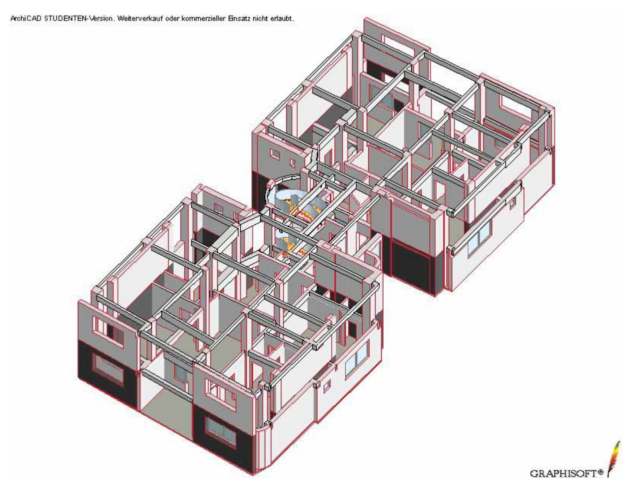


Figure 9: Seismic retrofit technique for "Interbelic" (from Bostenaru[4])

Fictive models

Two different blocks of flats (fig. 10-15), designed according to the same urbanistic rules and architectural use as model "Interbelic", regarding dimensions of spaces for different functions, usage mix, the type of connection between spaces, were assessed.

Figure 10: Perspective of the structure of model "Calderon"

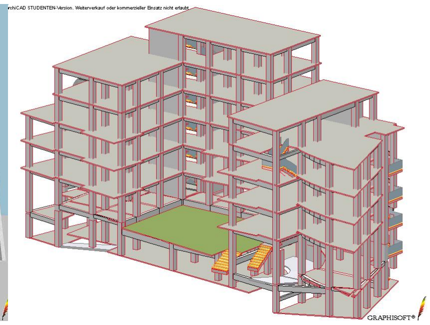


Figure 11: Axonometric view of model "Calderon"

Figure 12: Current floor of the building which served as model for "Calderon"

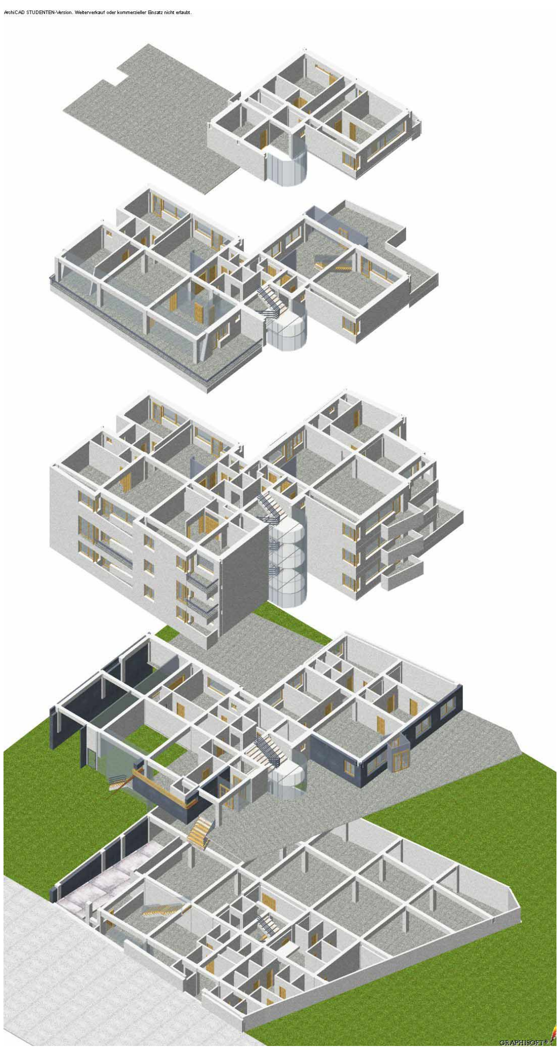


Figure 13: Axonometric view of all floors of the building which served as model for "Eminescu"

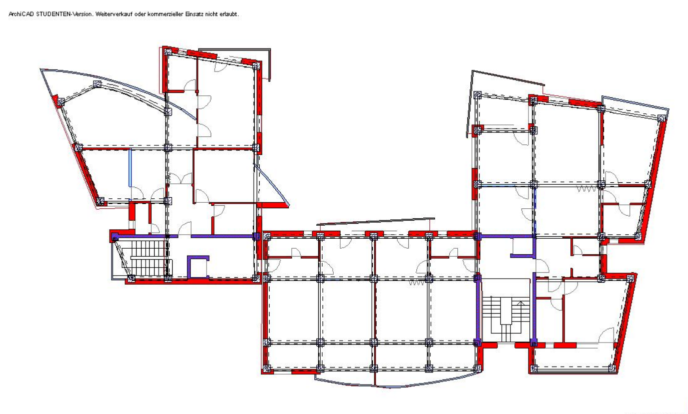


Figure 14: Perspective view of the building which served as model for "Calderon"

Figure 15: 3D section of the building which served as model for "Eminescu"

The urbanistic rules prescribed that above a cornice height of 24m each floor has to recess with at least 1.2m, to avoid street overshadowing. One parking place/flat had to be provided. The reason for doing this is that such flats might occur in the cityscape as the designs have been made for two real lots, or might provide alternative living space for people relocated during the retrofit measures in existing buildings, or might replace existing buildings with smaller historic significance.

Model “Calderon” consists of ground floor, basement and five upper floors, from which two present recesses (fig. 10-12 and 14). The building can be subdivided into a complex of two wings separated by a seismic joint: an L-shaped and a highly irregular one with four upper floors only, which additionally has a commercial higher ground floor. The slab over the basement has strong variations in vertical position, laying sometimes over a half-basement. In this case the retrofit was faked, as in the first analysis step the existing reinforced concrete structural wall was not modelled.

Model “Eminescu” consists of ground floor, basement and five upper floors from which the last two lay in recesses (fig. 13 and 15). The building is L-shaped, but within the building columns are distributed relatively regularly. The building has a huge basement, largely exceeding the size of the upper floors, half-height out of the ground. The heights are relatively constant, despite the commercial ground floor.

Within the two model buildings, called „Eminescu“ and „Calderon“, some deficiencies of the building type, like columns supported by beams only in upper floors, have been omitted. Also, the size of the buildings is bigger as in the previous example, but this represents no deviation from the typology.

APPLIED LOADING

Permanent loads 1.4 t/m^2 have been considered. These have been modelled as lumped masses at the upper end of the columns. For dynamic loads three different accelerograms have obtained from the European Strong Motion Database (Ambraseys[6]) and converted using SeismoSignal (SeismoSoft[3]), for the earthquakes in 1977, 1986 and 1990 (table 2). All three acceleration components of a „Vrancea“ earthquake had been applied simultaneously. The accelerograms as whole had been applied separately and successively as well. The records of the 1977 earthquake in station 39 (fig. 16-18) show the strength of the component in the vertical direction. For the strong ground movement in 1990, two selected records were employed, in order to emphasize the difference in peak acceleration and amplitude obtained at two different locations for the same earthquake. The duration of registered strong oscillations was different (24, respectively 50s) and, more important, the peak ground acceleration for the station 3186 was higher. Although the damages induced by the 1990 earthquake in the location of station 3186 were higher, they remained relatively small. The Thessalonica earthquake from the 20th of June, seemed to be suitable for the portability study purposes of this work for two reasons: first because many of the retrofit measures in this study are of Greek provenance and thus could potentially perform better on home structures and second because the same building type can be find in Bucharest and Thessalonica.

Table 2: Characteristics of considered earthquakes (data from Ambraseys[6])

earthquake		station location	building type	local geology	epicentral distance (km)	fault distance (km)	horizontal peak acceleration (m/s²)	vertical peak acceleration (m/s²)
Vrancea	1977	Bucharest INCERC (39)	free field	very soft soil	161	115	1.976	1.026
	1990				162	156	0.527	0.249
		Bucharest Măgurele (3186)	free field	-	174	169	0.896	0.595
	1986				134	121	1.355	0.503
Volvi 1978		Thessalonica City Hotel	structure related free field	soft soil	29	13	1.43	1.199

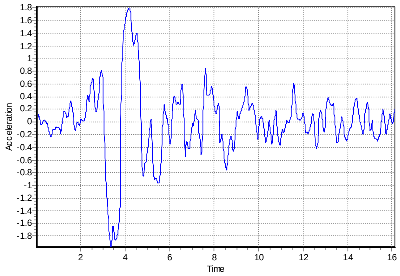


Figure 16: Accelerogram for the strong movement in 1977, epicentre in Vrancea, N-S direction (data from Ambraseys[6], software SeismoSignal[3])

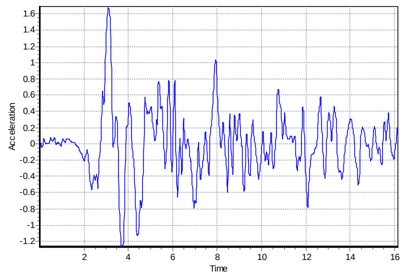


Figure 17: Accelerogram for the strong movement in 1977, epicentre in Vrancea, E-W direction (data from Ambraseys[6], software SeismoSignal[3])

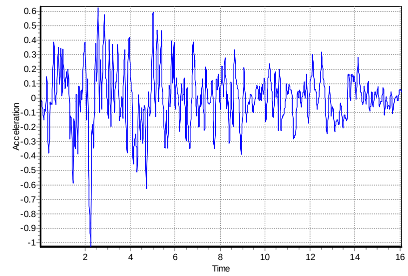


Figure 18: Accelerogram for the strong movement in 1977, epicentre in Vrancea, vertical direction (data from Ambraseys[6], software SeismoSignal[3])

STRESS-STRAIN BASED ANALYSIS AND INTERPRETATION

Performance criteria for gradual concrete and steel failure in the elements had been set and the time history of failure in the elements according to chosen criteria was computed. The results obtained were transformed into a table using the spreadsheet environment MS-Excel and then imported into a MS-Access database (fig. 19). The following types of progressive damage were considered:

- fracture of reinforcement bars together with cracking till crushing of concrete core, cracking and spalling of cover concrete;
- cracking and partial crushing of the concrete core, spalling and cracking of cover concrete, yielding of the reinforcement bars which are thus not protected any more;
- cracking and spalling of cover concrete, together with yielding of the reinforcement bars remained unprotected. The concrete core presents cracking only;
- cracking of the concrete of the members, till spalling. The reinforcement bars are not yielded yet;
- yielding of reinforcement bars together with slight damages in the concrete of the members. Yielding occurs most in beam members;
- cracking of both core and cover of the reinforcement concrete members;
- slight cracks in the concrete cover.

Using this, the numbers of elements presenting a particular damage could be counted, a transition from fine grid (ideal for computing strains) to coarse grid (ideal to visualise performance of building elements as a whole) made, the strength of damage assigned and from all these useful conclusions can be derived (fig. 22).

Figures showing comparatively the amount of damaged members reported to the total of structural members resulted into two tables: one for separate damage percentages and one showing the cumulative percentages. Table 3 shows the distribution of damage in percentages of structural members (columns and beams) damaged in the building. For those dictating the collapse mechanism the locations are shown in figure 20. These results for the left wing of model “Calderon”, showing the most damaged elements in the area where the wall was placed, show that the design wasn’t so bad. However, the performance of the building is poor even in the "retrofitted" variant, which shows that the sections of the members might need improvement. From the distribution it can be seen that the most heavy damage is presented by columns. However, in the columns mainly the concrete is damaged and yielding occurs more in the beams. It can also be read in which storeys the most damaged elements appear, which gives hints both to failure mechanism but also to the eventually necessary methods to perform reparation. An observation is that steel jacketing does not reduce cracking. The explanation might be that cracking still affects the upper part of structural members. Another observation is that the performance of the building is visibly improved by addition of side walls, although the side walls get damaged.

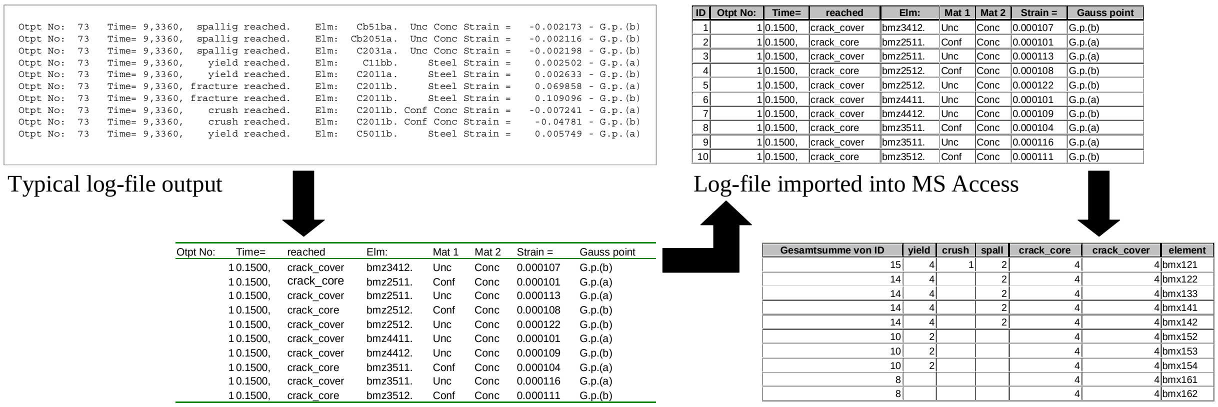


Figure 19: Workflow of data processing

Table 3: Percentages of elements showing cumulated damages in model "Eminescu"

Earthquake		Station	fracture	yield	crush	spall	crack core	crack cover
Volvi		City Hotel	0	9.77	8.65	9.77	67	73
Vrancea	1977	39	5	21.05	11.09	16.54	80	82
Vrancea	1986	3186	0	13.16	9.40	11.47	67	71
Vrancea	1990	39	0	8.46	0	4.32	50	56
Vrancea	1990	3186	0	12.41	8.08	10.15	56	64

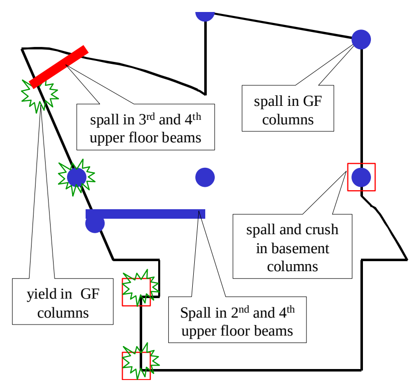


Figure 20: Most damaged elements in the left wing of model "Calderon"

DISCUSSION

Comparison to real damage types

For the general pathology description the example given by Penelis[7] was followed and the relevant information about column and beam failure (the ones computed in the simulations) extracted. Columns can be damaged due to cyclic flexure (predicted by the FE tool used) and low shear under strong axial compression. This leads to failure at the top and bottom, expressed in crushing of compression zones, successively on both faces (first spalling of concrete cover to reinforcement happens, then the core expands and crushes), associated with buckled bars in compression and hoop fracture. It is a very serious brittle type of failure. The column looses stiffness and ability to carry vertical loads. According to Penelis[7] it occurred at almost a quarter of the Thessalonica buildings in 1978 and after research of the author also in Romania. Another damage type for columns is failure due to cyclic shear and low flexure under strong axial compression. This one is expressed through X-shaped cracks in the weakest zone of the column. It occurs in short columns, usually ground floor and one sided masonry infill. This is a dangerous failure type, as vertical elements get destroyed, thus requiring immediate temporary support. It is not predicted by the numerical analyses and thus wasn't taken into account in this paper. Most common damage type for beams are cracks orthogonal to their axis along the tension zone of the span. These are to be explained through the vertical component of the seismic action, important both in Vrancea earthquakes and in the Thessalonica one. About almost half of the damaged beams is written to have suffered brittle shear failure near the supports in the Thessalonica earthquake (Penelis[7]), not predicted by the FE modelling employed again. For Bucharest no data concerning this were available. Both do not represent a threat for structure stability, though. More rare than shear failure are flexural cracks on the upper or lower face of the beam at supports. Shear or flexural failure can occur at the points where secondary beams or cut-off columns are supported by the beam under consideration, typically at skelet, not frame conformations like model “Interbelic”. X-shaped shear cracks can occur in short beams, like in model “Eminescu”, but are not dangerous for building stability. No such damage was observed in Bucharest in 1977. The largest part of the repair cost due to damage in infills as it implies repair in plastering. Although infill damage was been computed in simulations, the existence of infill was taken into account in the device directories. After the figures in Penelis[7] and taking into account the collapse mechanism for such buildings (Bostenaru[4]) it is to be assumed that damage of masonry infill always precedes damage of RC structural members. Large interstorey drifts also show the potential for extensive damage in infill system.

According to the simulations run for this work most serious damages occur at ground floor, reduced gradually in the upper floors in affected buildings in Bucharest but not in Thessalonica, which confirms the frequency analysis. Horizontally most damage appears in reality in areas which are far from the stiffness centre and mainly on the perimetre. As no slabs have been modelled for the analyses here, also the variation in stiffness of frames was not considered properly and thus the most damaged elements are predicted often in the middle. Vulnerability generally increases with the height of the buildings, but it also depends on frame characteristics, as this study showed. Most extensively models with six storeys were analysed here, which is the lower limit for extensive damage occurrence.

Comparison to real building types

A typological analysis regarding to what extend the models considered represent the Romanian building stock has been done. This analysis serves as a tool to make variations in the simple structures considered, in order to calibrate the structural results obtained for the further, economical studies. Common buildings, but especially buildings done by renowned architects were considered. The pictograms in figures 21 and 22 will show, without any scale, how plans of selected buildings look like. The selection has been made after criteria like number of frames in X and Y direction, spans and number of storeys. Although the most vulnerable Romanian buildings are seven to eleven floors high, the typological analysis shows that four storey multiple housing units have similar layout characteristics and were thus included in the analysis. The existence of recesses is not dependent of height, but much more on the use concept adopted in the configuration of the flats inside. The same applies concerning the regularity in column distribution.

The configurations of three selected buildings with four floors seem to be derived from that of model “Gregor”. The irregularity in the distribution of columns is small, but there are irregularities in floor shape, given by the existence of less filled frames and bays. Also the spans are very similar, but the height is different. A building with five floors of a type similar to model “Gregor” was investigated as well. The columns are regularly distributed, but both the shape in plan and the spans differ from the model. The details in the façade reveal the same style as in an example building in Thessalonica. Another five storey building with a 5x3 grid, like model “Özzi” has also an irregular distribution of columns in plan and in elevation. The real building considered in this study is within the height range of those with six floors. It has an irregular distribution of columns and a relatively irregular in floor shape (presenting two bays in the middle), but it can be categorised to be of 5x3 type, like model “Özzi”. To introduce irregularity in the “Özzi” structure in order to achieve closer results to those is an issue of further research. The same shape has been given to a plan derived from model “Özzi”, creating the bays in the middle and modifying the proportions between the spans of bays and frames. A next simplification will consider replacing the middle beams so they are not supported any more by beams, but by columns. The most heavy damage predicted is fracture of reinforcement. By this extent of damage already this idealised model of structure "Interbelic" comes close to the real one. An building designed by architect M. Janco (UAR[8]) within the same height range has a more regular distribution of columns coming close to the 5x4 grid of model “Gregor”. The spans vary with a short one in the long direction. Variations in height are minimal and on the last floor. With the seven floors range the spans become smaller, comparable to those considered for model “Calderon”. An building designed by architect D. Marcu (Marcu[9]) of this height considered in the typological analysis is also more svelte than the structural models. A eight floors building, designed also by M. Janco (UAR[8]), is an example of a well designed building from a point of view of structural conformation, with regular column distribution in the façade and only a slight irregularity in height, and still this building is listed risk category I (Lungu[10]). It is more svelte than the “Özzi” model, due to unequal spans in the two directions, but presents a similar grid to that one. Photos of the building side remained from the archive (UAR[8]) show what façade architecture can make out of a “regular structure” like those used for analysis in this work. The façade is narrow and decorated with an asymmetric balcony register, although this time the interior partition is almost symmetrical. Finally an early building designed by architect H. Creangă (UAR[11]) and one of his late chef-d’oevres of the same height resemble more or less the grid of the “Özzi” model, but with an enhanced irregularity in the distribution of internal columns and, in the first case, also regarding the spans and the external envelope. For the second one there is a joint in the middle of the building reducing each half to a size comparable to that of the models in this work.

Comparison between fine grid and coarse grid computed results

An observation concerns the shape of the response in the displacement time history depending on discretisation of the finite elements mesh. One element is considered for each structural member describes the coarse grid. Subdivision of structural members into elements define the fine grid. These topographies are used in order to optimise computation of results in FEM. Both “Gregor“ and „Özzi“ show in the analysis run on the course grid a slightly damped, but harmonic looking sinusoid (table 4). „Eminescu“ and “Calderon“ do show another type of response (fig. 23). There is first a big impulse, which damps to a more irregular but very low amplitude curve. When subdivision is introduced, the shape of the curve for „Calderon“ and “Eminescu“ remains the same. In case of „Calderon-left“, due to the highly irregular shape and the relatively big spans (many of them 6m), which in the small size of the building leaded to a small number of columns, the structural response has been bad. After about 10 seconds of dynamic solicitation, the structural load could not be applied anymore, as a displacement of 4.5m (equal to the average beam span) was reached! This can be seen as a „clean“ crash. The building would not survive the earthquake. This result is confirmed in the fine grid analysis. In the analysis of the retrofitted building it was clear that, despite numerical collapse, the structural collapse was predicted to be avoided. For „Calderon-right“ the response has been computed as being good. Only a maximum displacement of 150m is reached. The building is categorised, like „Eminescu“ to be earthquake safe.

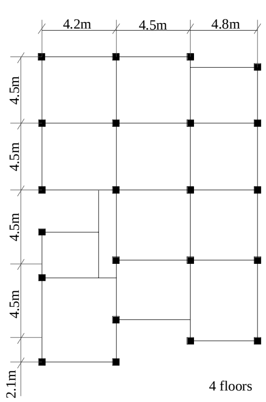


Figure 21: Scheme of mid-rise building plan

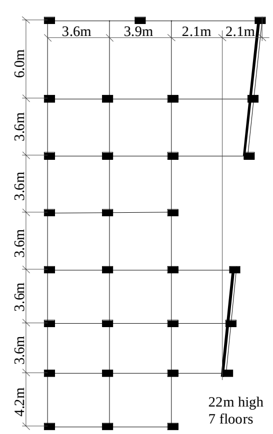


Figure 22: Scheme of high-rise building plan

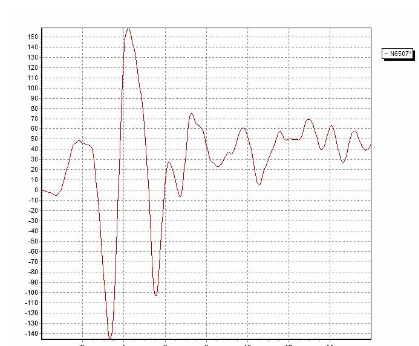
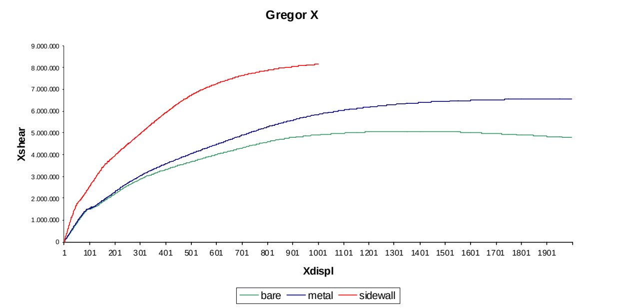
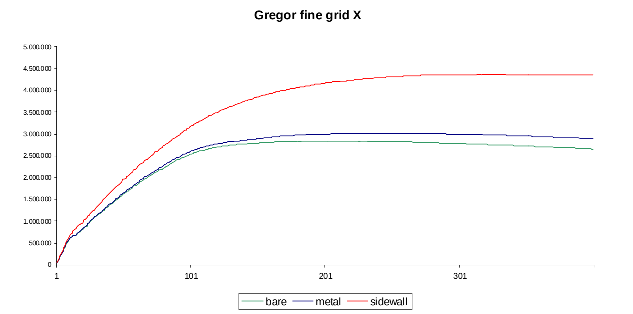


Figure 23: Displacement time history for model „Eminescu“, fine mesh grid, under the impact of the 1977 Vrancea earthquake, as predicted by analysis with SeismoStruct[3]

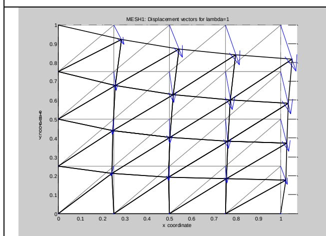
Table 4: Fine grid and coarse grid (after Bostenaru[4])



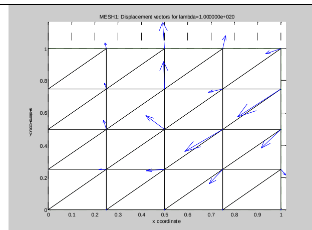
Pushover curves for model „Gregor“ under loading from X direction, coarse grid, bare structure and two retrofit possibilities.



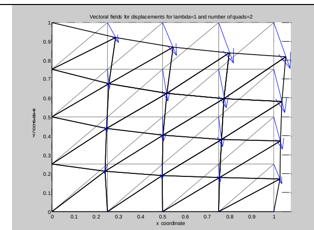
Pushover curves for model „Gregor“ under loading from X direction, fine grid, bare structure and two retrofit possibilities.



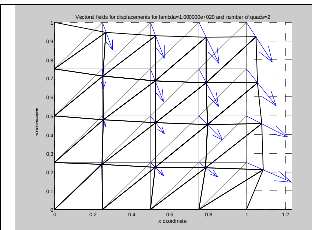
2D view of displacement vectors of a mesh without subdivision for $\lambda = 1$ and μ=1



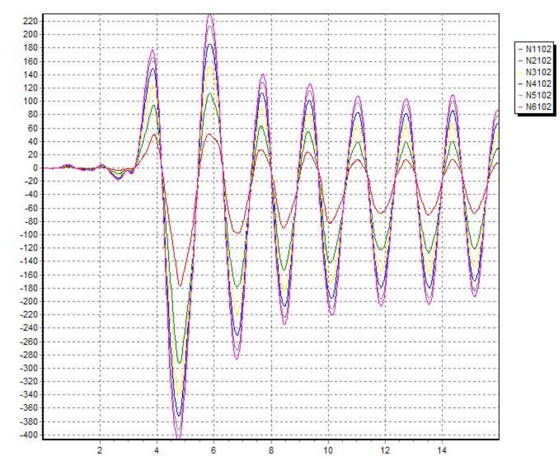
2D view of displacement vectors of a mesh without subdivision for $\lambda = 10^{20}$ und $\mu = 1$



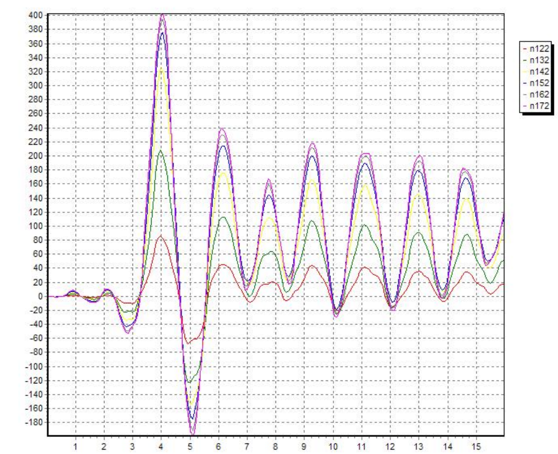
2D view of displacement vectors of a mesh generated with 2 quads for $\lambda = 1$ and μ=1



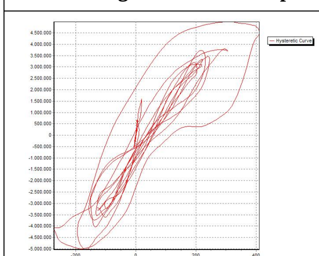
2D view of displacement vectors of a mesh generated with 2 quads for $\lambda = 10^{20}$ und $\mu = 1$



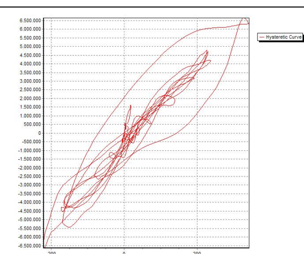
Drift time history in X direction as computed for model „Gregor“, coarse grid, under the impact of the 1977 Vrancea earthquake



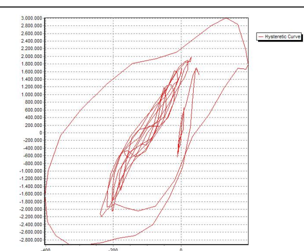
Drift time history in X direction as computed for model „Gregor“, fine grid, under the impact of the 1977 Vrancea earthquake



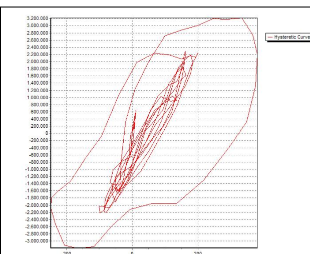
Hystereisis curve in the X direction, computed for model „Gregor“, coarse grid, under the impact of the 1977 Vrancea earthquake



Hystereisis curve in the X direction, computed for model „Gregor“, retrofitted with metal jacket, coarse grid, under the impact of the 1977 Vrancea EQ



Hystereisis curve in the X direction, computed for model „Gregor“, fine grid, under the impact of the 1977 Vrancea earthquake

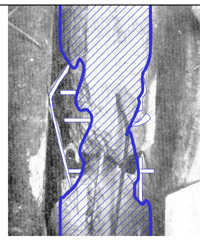


Hystereisis curve in the X direction, computed for model „Gregor“, retrofitted with metal jacket, fine grid, under the impact of the 1977 Vrancea EQ

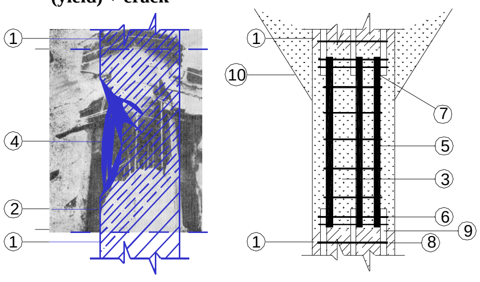
OUTLOOK TO ECONOMIC STUDIES

To each of the progressive damage types predicted in the simulation a “damage image” could be assessed. Such damages do no describe only the failure modes, but build the basis for the design of reparation measures. After taking into consideration all combinations of failing to meet certain performance criteria in steel or concrete sections of the structural members three basic types for each beams and columns were defined. Severe damage till concrete crush or reinforcement fracture requires partial replacement of the structural member in whole depth (fig. 24 and 25). For these a damage image, based on those from the Vrancea earthquake 1977 (Bălan[12]) and a detailed reparation approach, outgoing from the drawings in Penelis[7] for columns, but for both columns and beams a detailed to device directory level were provided. The costs for the reparation of all kind of damages, like also the costs for retrofit presented before, were calculated using material prices and salary values in Germany.

COLUMN: FRACTURE + crush + spall + (yield) + crack



COLUMN: CRUSH + spall + (yield) + crack

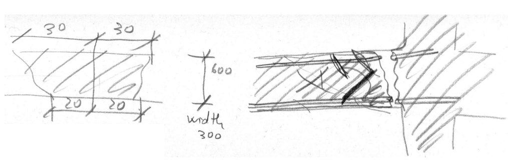


1. old concrete, 2. damaged concrete, 3. new concrete
4. buckled reinforcement, 5. new reinforcement, 6. new stirrups,
7. welded joint, 8. old stirrups, 9. old reinforcement, 10. formwork

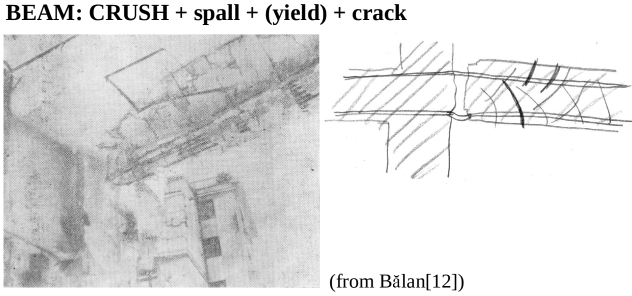
No.	Work	Unit	time /unit	price/ unit (€)	No. units	price/ time (€)	total (€)
1	Breaking up masonry around the columns	m³	5.6h	-	0,26	36	52.41
2	Unloading the column through bolts	piece	3h	-	1	36	108
3	Breaking up concrete with chipping hammer	m³	25h	-	1	36	900
4	Cleaning up removed concrete	m³	-	13+6	1	-	19
5	Cutting away damaged zones of reinforcement	piece	2 min	-	9	36	12
6	Laying new reinforcement	kg	60/h	-	136	36	81.6
		kg	-	0.5	136	-	68
7	Air blasting of concrete&reinforcement	m2	4h	-	0.32	36	46.08
8	Making and fixing new stirrups	kg	60/h	-	134	36	80.4
		kg	-	0.5	134	-	67
9	Anchoring stirrups to the longitudinal re-bars	piece	2.5h	-	62	36	90
10	Concrete surface roughening (sand blasting)	piece	2h	-	1	36	72
11	Setting up formwork	piece	2h	-	1	36	72
	Formwork	m²	-	8.5	10	-	85/4
	Formwork support (scantling 4/6)	m	-	0.55	25	-	13.7/4
12	Casting concrete	m³	1h	-	1	36	36
		m³	-	100	1	-	100
13	Removing formwork	piece	2h	-	1	36	72
14	Plastering (interior and exterior)	m	0.5h		6.25	36	225
	PRICE						2126

Figure 24: Damage image, reparation approach and device directory for repairing severe column damage (after Bostenaru[4])

BEAM: FRACTURE + crush + spall + (yield) + crack



BEAM: CRUSH + spall + (yield) + crack



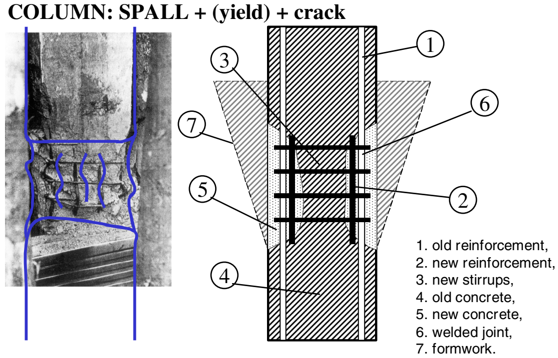
(from Bălan[12])

No.	Work	total price (€)
1	Removing plastering	32,4
2	Removing flooring	
3	Reducing deflection	36
4	Breaking up concrete with chipping hammer	900
5	Disposing the ablated material	19
6	Cutting away damaged zones of reinforcement	9
7	Making slab breakthroughs (10x10cm)	180
8	Air blasting concrete&reinforcement	51,84
9	Cleaning the surface	25,92
10	Laying new reinforcement	81,6
	Steel	68
11	Making and fixing new stirrups	80,4
	Steel	67
12	Anchoring stirrups to longitudinal reinforcement	90
13	Setting up formwork	72
	Formwork	7,65/4
	Formwork support (scantling 4/6)	1,32
14	Casting concrete	36
	Concrete	100
15	Removing formwork	72
16	Plastering	13,5
	Plaster mortar	1,875
17	Repairing flooring	
	PRICE	1938,88

Figure 25: Damage image, reparation approach and device directory for repairing severe beam damage (after Bostenaru[4])

Moderate damage till spalling of cover concrete needs local superficial repair (fig. 26 and 27) For these a damage image was compiled, based on those from the Vrancea earthquake 1977 (Bălan[12]) and a detailed reparation approach, outgoing from the sketches in Penelis[7] for columns and from NTUA[13] (drawings by Bourlotos[14]) for beams. In both cases a detailed to device directory level were provided.

COLUMN: SPALL + (yield) + crack



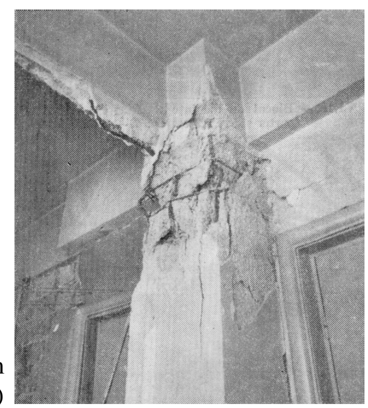
No.	Work	total price (€)
1	Breaking up masonry around the columns	52,41
2	Breaking up concrete with chipping hammer	300
3	Cleaning up broken concrete	6,33
4	Cutting away damaged zones of reinforcement	12
5	Laying new reinforcement	149,6
7	Air blasting concrete&reinforcement	684
8	Making and fixing new stirrups	80,4
		67
9	Anchoring of stirrups to longitudinal reinforcement	90
10	Setting up formwork	72
	Formwork	21
	Formwork support (scantling 4/6)	3,58
11	Casting concrete	45,33
12	Removing formwork	72
13	Plastering (interior and exterior)	54
	PRICE	1709,76

Figure 26: Damage image, reparation approach and device directory for repairing moderate column damage (after Bostenaru[4])

BEAM: SPALL + (yield) + crack

No.	Work	total price (€)
1	Reducing slab deflection	36
2	Removing concrete cover	288
3	Disposing the rubbish	0,1368
4	Treatment of the steel	144
5	Laying new reinforcement	0,2592
	Steel	0,2
6	Making and fixing new stirrups	0,2592
	Steel	0,2
7	Anchoring of stirrups to the longitudinal reinforcement	180
8	Casting concrete (Shotcrete)	144
	Concrete	1,26
9	Removing rebound	36
10	Disposing rebound	0,95
11	Plastering	32,4
	Plaster mortar	0,45
	PRICE	864,11

(from Bălan[12])



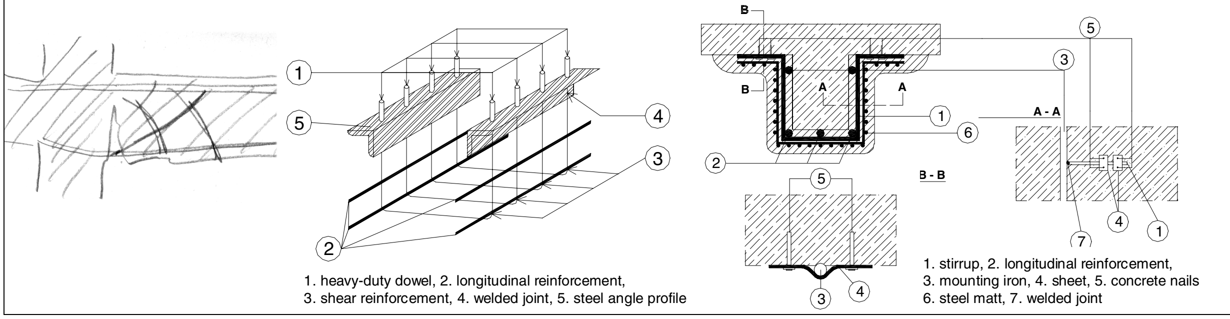


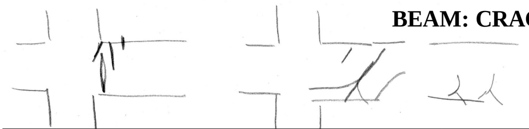
Figure 27: Damage image, reparation approach and device directory for repairing moderate beam damage (after Bostenaru[4])

For slight damage through cracking repair consists usually out of epoxy resins injections. Thus besides of a damage image sketch only the device directory for the reparation has been provided (fig. 28 and 29). For all levels of damages, the repair costs for beams and columns vary only slightly. Thus a unitary repair cost depending on the specific cumulated damage only could be assigned and multiplied by the number of elements presenting this, being columns or beams.

COLUMN: CRACK

No.	Work	total price (€)
1	Removing plastering	129,6
2	Cleaning the surface	172,8
3	Mechanical drilling of 2cm injection holes	86,4
4	Cleaning drill holes	86,4
5	Driving in the packer	86,4
6	Isolating	86,4
7	Verifying the throughfare	86,4
8	Turning in the nipples	86,4
9	Injection	86,4
	Injections means	24
10	Post-injection	86,4
11	Removing isolation	86,4
12	Driving out the packer	86,4
13	Closing the drill holes	86,4
14	Disposing the plastering broken up	0,46
15	New plastering	60,48
	Plastering mortar	12
	PRICE	1349,74

Figure 28: Damage image, reparation approach and device directory for repairing slight column damage (after Bostenaru[4])



BEAM: CRACK

No.	Work	total price (€)
1	Remove plastering	135
2	Cleaning the surface	180
3	Mechanical drilling of 2cm injection holes	90
4	Cleaning drill holes	90
5	Driving in the packer	90
6	Isolating	90
7	Verifying the throughfare	90
8	Turning in the nipples	90
9	Injection	90
	Injection means	33,75
10	Post-injection	90
11	Removing isolation	90
12	Driving out the packer	90
13	Closing the drill hole	90
14	Removing the broken up plastering	
15	New plastering	90
	Plaster mortar	16,87
	PRICE	1355,62

Figure 29: Damage image, reparation approach and device directory for repairing slight beam damage (after Bostenaru[4])

Using the results from the structural assessment the costs for repair for the whole model were computed. Adding the costs assigned for retrofit, computed on the basis presented in the paragraph on modelling, the total costs for retrofit and repair were computed. In case of successively applied earthquakes a more expensive retrofit in the earthquake preparedness phase can lead to a decrease in the amount of damage and subsequent repair costs, as shown in the diagrams in figures 30 and 31 for models "Gregor" and "Özzi" and various retrofit options.

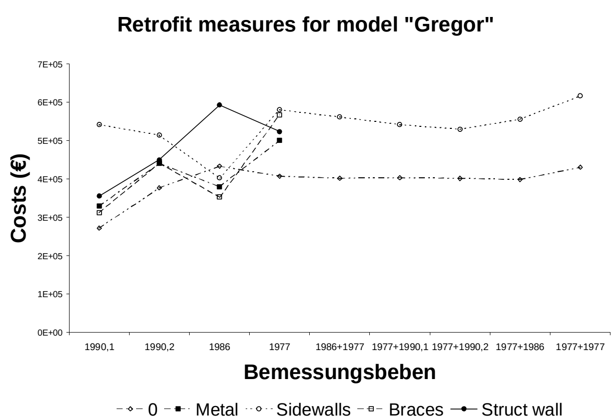


Figure 30: Costs curves for retrofit type alternatives

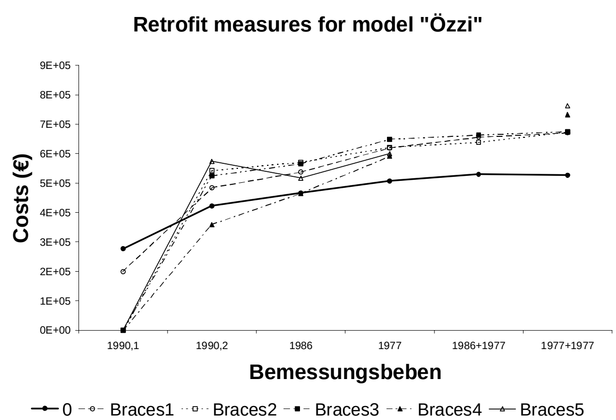


Figure 31: Costs curves for retrofit layout alternatives

Further, connection with the concept of performance levels (SEAOC[15]) was necessary. In figure 32 the progressive damage states are shown related to a typical pushover curve. Certain displacement values describe the likelihood of reaching certain material stress-strain values. Typically, the "size" of the earthquake considered is given on the vertical axis of base shear. However, the German term of "Bemessungsbeben" (Paulay[16]), meaning design earthquake, can be also associated to the displacement performance levels, as it defines the earthquake at which such a state has to be provided for.

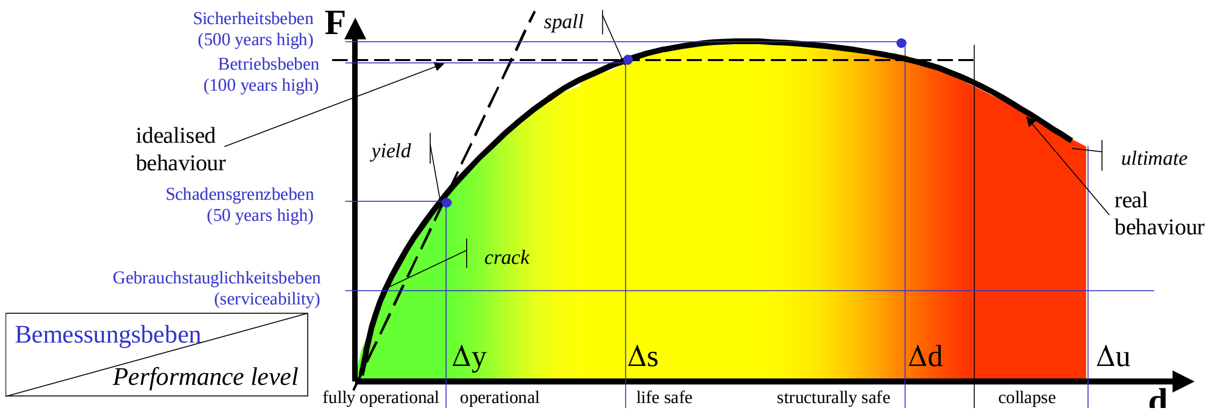


Figure 32: Structure limit states

CONCLUSIONS

This paper showed an innovative algorithm of how to use structural assessment and other computer software in order to obtain results of use in interdisciplinary efficiency studies. In order to assure best portability of the methods the models have been kept as location neutral as possible , although, of course, their characteristics, used in the study, are determined by it. This stress-strain based methodology to assess structural performance have already proved useful for economic studies and was further used in the author's doctorate dissertation. The displacement based interpretation of the time history analysis and the static pushover analysis served only the assessment of the reliability of the discretisation degree.

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