Session 8: Assessment and rehabilitation of existing buildings after accidents

Objekttyp: **Group**

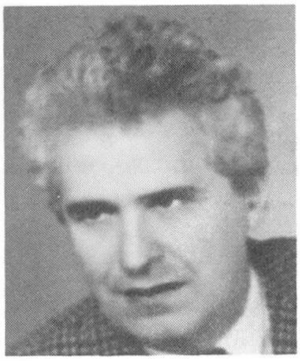
Zeitschrift: **IABSE reports = Rapports AIPC = IVBH Berichte**

Band (Jahr): **77 (1998)**

PDF erstellt am: **22.07.2024**

Seismic Rehabilitation of Existing Buildings in Romania

Liviu CRAINIC
Professor
Techn. Univ. of Civil Engineering
Bucharest, Romania



Liviu Crainic, born 1938, received his civil engineerng degree from the Technical University of Cluj/Romania (1960) and his PhD in 1974, from Technical University of Construction Bucharest. He is currently professor, head of the Department of R/C Structures of TUCB.

Summary

The preservation of existing buildings in Romania involves accurate *seismic assessment* and implementation of appropriate methods for their *repair/strengthening*. An analytical procedure for assessment of r/c structures is presented herein and its role in seismic behavior investigation and identification of structural system weak components is discussed. Procedures for selecting appropriate upgrading solutions, currently implemented in Romania, are briefly described. Examples illustrate the above considerations.

1. Static Post-Elastic Procedure for Seismic Capacity Evaluation

The present article proposes a *static post-elastic procedure* for determining the seismic shear force capacity. Incremental loading with *imposed displacements*, similar to those generated by seismic action, is considered. Thus, the *structure capacity degradation,* due to progressive elements failure, and *the weak structural components,* are stressed out. This information is turned into relevant input data for selecting the appropriate *rehabilitation solution.*

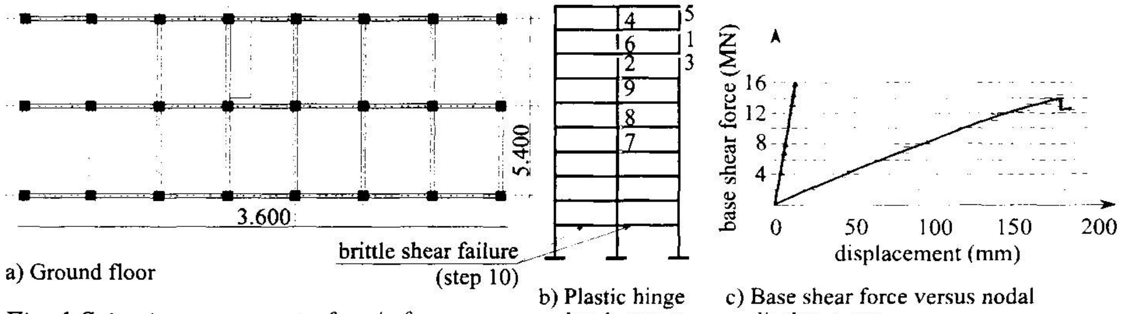
The following steps define the procedure:
(i) Determine internal forces in critical sections due to gravity loads $\{S_g\}$ and to seismic equivalent forces $\{S_E\}$.
(ii) Determine lateral displacements of the structure, due to seismic forces $\{\Delta_E\}$.
(iii) Calculate the moment capacity of critical sections $\{M_{cap}\}$.
(iv) For each critical section *i* determine the coefficient $\gamma_i = M_{E,i} / (M_{cap,i} - M_{g,i})$.
(v) Select maximum magnitude of γ_i : $\max\{\gamma_i\} = \gamma_j$. In section *j* a plastic hinge will occur.
(vi) Determine the horizontal seismic force (basic shear force) $F^{(1)}$, internal forces $\{S_E^{(1)}\}$ and lateral displacements $\{\Delta_E^{(1)}\}$ corresponding to the plastic hinge occurrence:
$F^{(1)} = F/\gamma_j$; $\{S_E^{(1)}\} = \{S_E\}/\gamma_j$; $\{\Delta_E^{(1)}\} = \{\Delta_E\}/\gamma_j$
(vii) Check-up the magnitude of plastic rotation and shear force in critical sections
- plastic rotation and/or shear force within accepted limits → continue
- excessive plastic rotation and/or shear force → member failed → delete failed member from structure → go to (i) (with new input data, considering the structure *without* the failed member).

(viii) Determine the stiffness matrix of the structure, considering a plastic hinge in section *j*
(ix) Increase lateral displacements with an accepted increment.
(x) Calculate internal forces corresponding to displacements of step (ix)
(xi) Go to (iii).

2. Seismic Rehabilitation

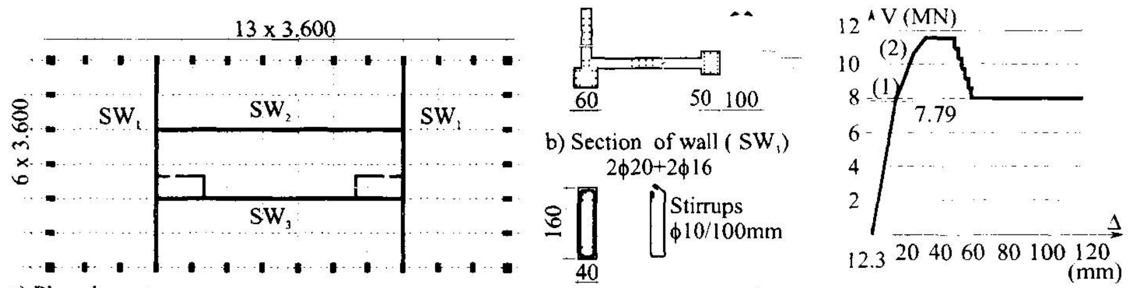
The selection of a seismic rehabilitation strategy and decision has to take into account: the potential or existing damage and failure of structural and non-structural components, and other similar factors. Basically, the rehabilitation has to rectify "weakness" at both local and overall scale of the building.

The seismic rehabilitation solutions for reinforced concrete structures, currently implemented in Romania are: (a) *jacketing* of existing elements and (b) *addition of new structural elements,* especially structural walls. The adopted solution depends upon the *diagnosis* resulted from in-situ inspection ("qualitative assessment"), analytical evaluation and cost-benefit analysis.



a) Ground floor
b) Plastic hinge development
c) Base shear force versus nodal displacement

Fig. 1 Seismic assessment of a r/c frame



a) Plane layout
c) Coupling beam detailing
d) Base seismic shear force V versus displacement Δ

Fig.2 Seismic behavior of a dual r/c structural system

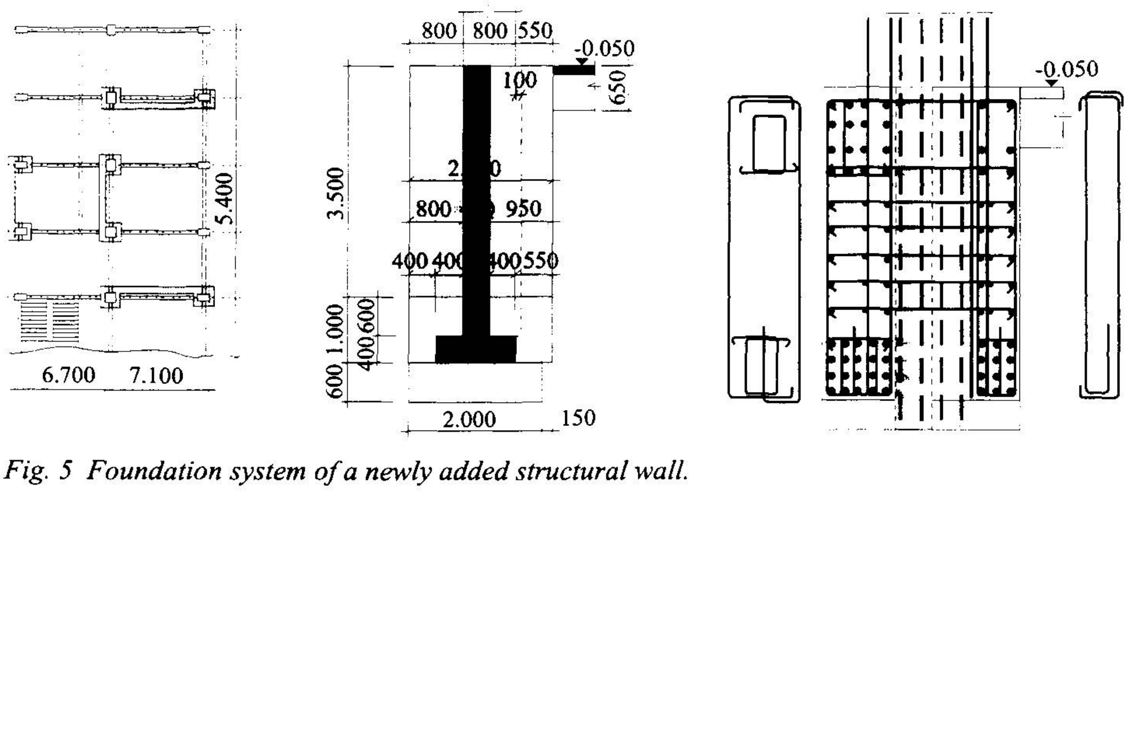
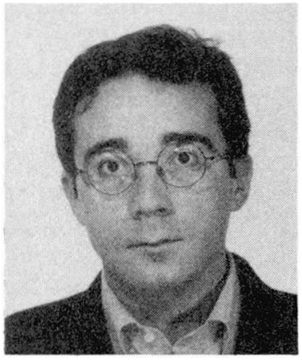


Fig. 5 Foundation system of a newly added structural wall.

War and Earthquake Damage in Dubrovnik: Preventive and Remedial Measures

Giorgio CROCI
Professor
University of Rome
Rome, Italy

Alessandro BONCI
Civil Engineer
University of Rome
Rome, Italy



Giorgio Croci, born 1936, has carried out much research into, and many important projects for the restoration of historical buildings such as the Colosseum and the Senatorio Palace in Rome, the Tower of Pisa and, following the recent earthquake in Assisi, the Basilica of S. Francis. He is a consultant for the Council of Europe and for UNESCO.

Alessandro Bonci, born 1968, graduated in civil engineering at the University "La Sapienza" in 1995. Since then he has been involved with various restoration projects and as an adviser for UNESCO. He's now working on UE research concerned with new technologies for the seismic protection of historic buildings

Summary

In the historic towns of Dubrovnik and Ston, building dating back up to the 14^{th} century, , suffered major damage during the recent war and were then subsequently struck by a strong earthquake in September 1996; Because of the exceptional value of the sites, UNESCO organised several missions which involved Prof. G. Croci. This paper summarises the studies carried out after a detailed survey which sought to examine the structural behaviour of the buildings and the relationship between local construction techniques and the actual damage. With this as a basis, various guidelines for preventive and remedial measures have been proposed with specific reference to the local building methods and typologies.

Keywords: Croatia, war, earthquake, historic buildings, damage, prevention, remedial measures

1. The characteristics of the historic buildings and the surveyed damage

The main walls of the buildings of Dubrovnik and Ston are usually made of sack masonry, with the timber floors and roofs supported by stone cantilevers inserted in the walls (Fig. 1); this structural solution, because of the lack of connection to the walls, does not help develop the co-operation among the walls themselves, which would allow them to resist the horizontal forces more effectively.
In Dubrovnik the signs of shell impacts are still visible on the fronts of the houses: direct hits struck about 70 % of the 824 buildings of the inner city. Further damage was produced by the fires following to the explosions (Fig. 1): the roofs of 5 buildings were affected by fire and in 9 buildings the roofs and floors were completely destroyed.
The damage caused by the earthquake of September 5^{th} 1996 (peak ground acceleration of 0.6 g, epicentre 10 km SE from Ston) can be traced, however, to other causes. These are:

Intrinsic weakness of the sack masonry (weak bond, decay of the mortar):which explains the detachment of blocks in the external skin and the collapse of entire sections of wall. (Fig. 2).
Structural weakness of the walls: in the façades which were almost parallel to the seismic action (left façade in Fig. 3) cracks under the windows, due to the insufficient stiffness of the floor bands (reduced thickness of the zone under the windows, the lack of horizontal ties), were frequently noted; the façades almost perpendicular to the seismic action were often detached due to the inadequate connections to the lateral walls (left façade in Fig. 3).
Lack of connection between roofs and floors with the walls, which not only reduces the co-working of the walls, but sometimes leads to the slippage of the roofs themselves.

Fig. 1

Fig. 2

Fig. 3

2. Preventive and remedial measures

The rigid application of the Croatian Seismic Code regulations would have required the large use of reinforced concrete elements; we succeeded however in following a different philosophy, using where possible, the traditional constructional techniques.

The materials: the intrinsic weakness of the sack masonry construction requires, both as a preventive and a remedial measure, the injection of the walls, or, in the case of very loose material, which was very common, the use of grout percolated in from the top; cracks, depending on their width, could be filled with grout or repaired with the technique of "unpicking and sewing".(toothing out and bonding)

The walls: different measures could be adopted depending on the characteristics of the building.
a) Insertion of horizontal ties at floor level to connect the walls together.
b) Construction of a kind of "kerb" (Fig. 4_a), using various possible technical solutions (Fig. 4a', 4a"), at the level of the roof to connect the upper part of the walls (fig. 4_b) and to connect the roof to the main walls (Fig. 4_c).

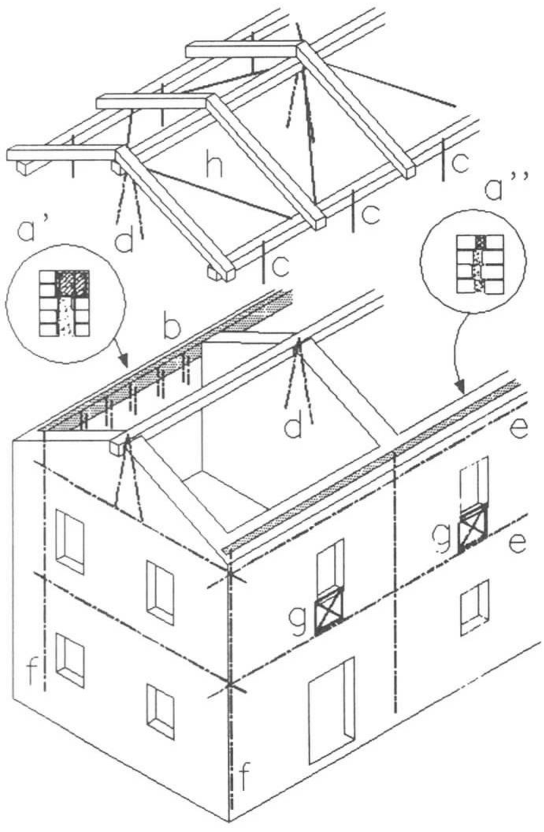


Fig. 4

Load Testing of Large Panel System Dwellings

Barry REEVES
Senior Research Engineer
Building Research Establishment Ltd
Watford, UK

Barry Reeves, born 1955, received his civil engineering degree from the Royal Military College of Science, Shrivenham in 1981. He is a member of the Construction Division at the Building Research Establishment Ltd, involved in the appraisal and long-term monitoring of structures.

Summary

This paper describes the programme of full-scale static proof load tests carried out by BRE on a 10-storey block built in the late 1960's using the REEMA Conclad large panel system (LPS) of construction. The load tests were aimed at establishing the performance of selected areas of the building under the currently employed notional loading criterion of 17 kN/sqm. This was taken to be acting as an equivalent static pressure applied simultaneously to the surfaces of all structural elements bounding the enclosed space containing the explosion. The load tests have conclusively shown that the Reema Conclad block tested, and a Bison system built block tested earlier in the year, are able to accommodate the minimum specified notional loading of 17 kN/sqm without gross distortion of the panels and accompanying joints. These were therefore sufficiently strong to resist the accidental loads associated with non-piped gas explosions.

Keywords : Concrete, panel, static loading, explosion, large panel system, LPS

1. Introduction

A Tribunal was set up to investigate the cause and implications of the partial collapse which occurred in 1968 of a 23-storey large concrete panel system (LPS) built tower block built in London. The block was built using the Taylor Woodrow-Anglian system of LPS construction. The Tribunal recommended that system-built blocks of flats over 6-storeys in height should be appraised and, if needed, strengthened. In considering whether strengthening was required engineers were to consider the affects of forces on the structure equivalent to a static pressure of 34 kN/sqm where piped gas was supplied to the building, and 17kN/sqm where it was not.

Unless there are particular problems with the workmanship in a block, the current view held by BRE is that extensive strengthening is unlikely to be necessary for many types of block without a piped gas supply. However, up until now there has been no way of establishing this definitively because of the lack of information on the performance under accidental loads of LPS buildings without a piped gas supply. Consequently, experimental data was required to resolve these issues and to provide engineers with a better basis for undertaking structural appraisals of these buildings.

To meet these needs, the Building Research Establishment Ltd (BRE) obtained temporary possession of a 20-storey BISON and a 10-storey REEMA Conclad LPS block prior to their demolition. A series of load tests were carried out in the two buildings during 1997. Provisional results of the load tests carried out in the Bison LPS block have been reported previously.

The test programme for each block was carried out in two phases. Phase I consisted of structural investigations of the block, with Phase II involving a series of static load tests carried out in selected areas of that block. Those for the Reema block are described below.

2. Phase I - Investigation of the form and quality of construction

The preliminary investigations were aimed at verifying that the design intentions had been complied with and assessing the standard and consistency of workmanship within the building. Consequently, selected joints were opened up within rooms away from the zones in which the main load tests where to be performed.

Fig. 1 External view of a Reema Conclad block

The quality and consistency of workmanship within the building was, in the main, found to be reasonably good in all the areas opened-up, although there was some variation in the detailing of the tying reinforcement.

Therefore, the structural performance of the areas load tested was likely to be representative of similar areas in other nominally identical Reema blocks located elsewhere in the country.

3. Phase II - Static load tests

The load tests were aimed at investigating the ability of structural elements within the block to withstand the notional loads associated with the UK accidental loading criterion of 17 kN/sqm. The maximum mid-span bending moment and maximum shear force at the supports which would result from the uniformly distributed loading induced by such a loading, was applied simultaneously to the wall and floor components using a system of hydraulic jacks, adjustable lightweight loading shores and steel distribution beams.

Several forms of protection were provided to prevent excessive movement of the wall panels and floor slabs in the event of a premature failure of any of the components under test. These consisted of Acrow props, tie rods (loose during the test) fixed between opposing wall panels and steel fixing angles bolted (loose during the test between the floor slabs and top/bottom of the walls.

The physical response of the structure to the applied loads was measured using displacement transducers mounted on an independent instrumentation frame and vibrating wire strain gauges attached at selected positions on the floor slabs.

4. Conclusions

The static load tests have shown that the Reema Conclad block tested by BRE (and the Bison block tested earlier in the year) was able to accommodate the minimum specified notional loading of 17 kN/sqm without gross distortion of the panels and accompanying joints. Accordingly, the Reema (and Bison) buildings tested are judged sufficiently strong to resist the accidental loads associated with non-piped gas explosions.

It has been demonstrated that static load testing might be used to assess the safety of particular designs of LPS buildings where calculations suggest that a structure may have an inadequate margin of safety for the specified notional accidental loading case.

Strengthening of Buildings according to Eurocode 8 and National Standards

Roko ZARNIC
Assistant Professor
University of Ljubljana
Ljubljana, Slovenia

Vlatko BOSILJKOV
Research Assistant
University of Ljubljana
Ljubljana, Slovenia

Martin BAUMGARTNER
Civil Engineer
Civil Engineering Institute ZRMK
Ljubljana, Slovenia

Summary

The European codes are currently being introduced in Slovenia. Eurocode 8 has already been accepted as a parallel code to the existing national codes for structural design in the earthquake prone areas. Therefore, the need for comparison between the new and the existing codes through the case studies is one of the most important issues in the current Slovenian practice. The paper presents a case study of a mixed reinforced concrete and masonry five-storey building constructed in Ljubljana in 1907. The parametric analysis of the earthquake resistance has been performed by the inelastic pushover analysis. The present configuration of structure is compared to gradually upgraded configurations. The comparison of the load-bearing capacity of the upgraded structure and the demands given by Eurocode 8 illustrate the problems of strengthening historic buildings.

1 Slovenian national standards and Eurocode 8

Slovenia is in a transition period and some codes of the former Yugoslavia are still in active use. After the independent state had been established in 1991, it was decided to adopt the European codes as Slovenian with the needed modifications related to the local circumstances. Therefore, the entire Eurocode 8, including its Part 1.4, are to be a base for the future Slovenian code. Presently Eurocode 8 has been accepted as Slovenian prestandard for an experimental application period. However, in this transition period the old 1988 Yugoslav code is still valid. Eurocode 8, Part 1-4 gives the major guidelines and criteria for the design of building in seismic areas, and they also influence the rehabilitation works. The major dilemma is, how strictly should the existing buildings be treated in the process of rehabilitation. The fact is that older masonry and reinforced-concrete structures were not constructed according to any seismic code. The strict respect for the existing codes increases the investments in the rehabilitation or even makes them restricted in the cases of valuable cultural monuments. The case study, presented in the sequel, illustrates the problems of decision making regarding the future use of a historic building from the beginning of this century.

2 Case study

The building was erected in 1907, twelve years after the last strong earthquake in Ljubljana (Fig. 1). In the future the building will be adapted for the university use. The building is 54.8 m long and 20.8 m wide. It has five storeys and a basement. The heights of the first, second, third, forth and fifth storey are 4.5 m, 3.9 m, 3.9, 3.5 m and 2.6 m, respectively. The main structural system of the building consists of clay-brick masonry walls, reinforced concrete columns and reinforced concrete slabs. The slabs were designed to carry live load of 20 kN/m^2. The masonry walls and columns are wider in the lower storeys. In the first storey the masonry walls are 0.75 m and 0.60 m wide while in the fifth storey their width is 0,40 m. After having analysed of several configurations of plan layouts, the one that is shown in Figure 2 was chosen. It was a compromise between the structural and the future user demands.

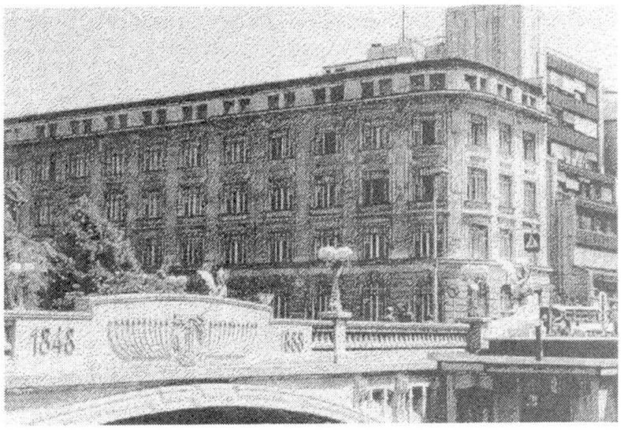


Figure 1 The building build in 1907

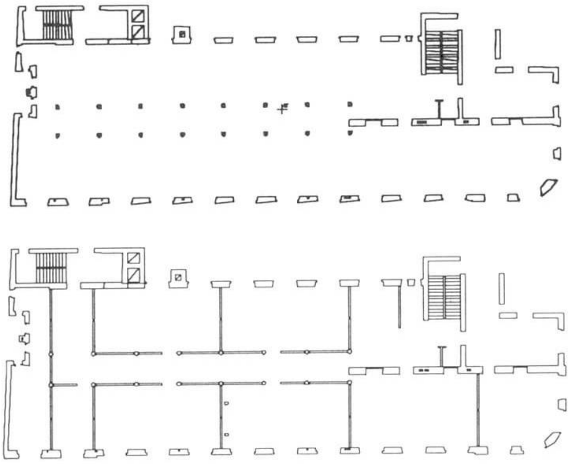


Figure 2 The plan of the existing and strengthened 1^{st} floor

The earthquake resistance analysis is based on inelastic method. In Figure 3 the storey diagrams calculated for the weaker direction of building are compared.

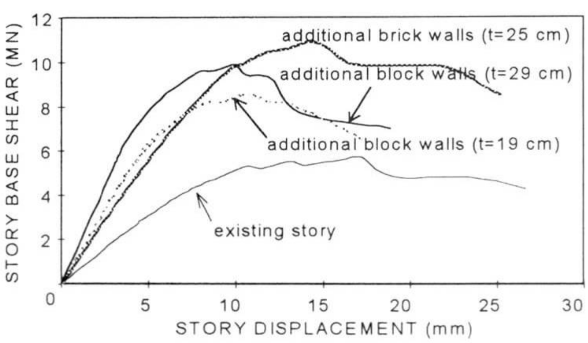


Figure 3 Comparison of storey base shear diagrams in the case of confined walls

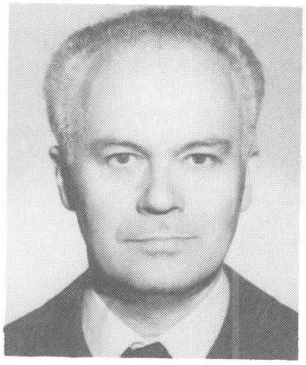
The influence of additional walls on the existing structure caused the lowering of torsional effects. The results of the analysis clearly show the importance of proper selection of materials and confinement of walls. It can be concluded that the most effective walls can be build from solid clay brick masonry that is less stiff than hollow block masonry of the same strength. Therefore one of the main tasks in the structural design of strengthening is selection of suitable materials. They should be compatible to the original materials as it can be seen from this case study.

In the national codes the safety demands are differentiated according to use of each building. Importance category I is demanded for public buildings of higher importance (hospitals, schools etc.). Residential buildings and less important office buildings are considered as buildings of importance category II. The herein-described building can be strengthened by masonry to meet the demands of the national codes for the buildings of category II. If the same building would be used in the future for more demanding purposes, another strengthening technique should be necessary. It will be anyway the case when Eurocode 8 comes in force as the national code.

The existing built heritages in European towns that are located in earthquake prone areas generally do not fulfil the demands given in contemporary codes. It is to be discussed in which extend and with which techniques the structures can be strengthened. . In the process of strengthening a special attention should be paid to the selection of materials. Their characteristics should be compatible with the characteristics of the originally used materials.

Saving Buildings Under Conditions of High Seismic Risk

Horea SANDI
Research Advisor
INCERC
Bucharest, Romania



Horea Sandi, born 1932, received his civil engineering degree and his doctoral degree from the Inst. of Civil Engineering, Bucharest, and his mathematics degree from the Univ. Bucharest. Humboldt fellow at the Technical Univ. Hanover, 1968-69. Vice-president of EAEE, 1982-1986.

Summary

Some specific features of the problem of saving buildings in areas affected by high seismic risk are dealt with. The case of Romania, especially of its part affected mainly by Vrancea earthquakes, is considered a relevant case in this view. Some data on the seismic conditions and on vulnerability and risk affecting buildings of Romania are given. The development of risk reduction strategies is then dealt with. The specific framework and the development of cost-benefit analyses are discussed.

Keywords: seismic risk, risk mitigation, vulnerability, cost-benefit analysis.

1. Introduction

The intention to save buildings in areas affected by high seismicity, which exist in Romania as in some other countries of Central and Eastern Europe, must face the additional problems raised by a high seismic risk which affects, according to experience and to knowledge at hand, extensive parts of the existing building stock. The additional problems raised by seismic risk are highly acute. The need to reduce the seismic risk that affects in Romania, as elsewhere, important parts of the existing building stock, may easily become the highest priority as compared with other aspects related to the preservation of this heritage, that are common to various countries or regions.

2. Some data on the state of the art in Romania

The natural seismic conditions are determined by the existence of several source zones. Out of these, the intermediate depth Vrancea source zone, located outside of the bow of the Carpathians, delivers more than 95 % of the energy delivered in the average per century in Romania. This source zone affects with high intensities several times per century extensive parts of the territory. The return periods of Gutenberg-Richter magnitudes are in the range of 6 years for $M = 6.$, 14 years for $M = 6.5$, 32 years for $M = 7.$, 46 years for $M = 7.2$, 82 years for $M = 7.4$. 126 years for $M = 7.5$, 234 years for $M = 7.6$ etc. (the strongest magnitudes observed in this century were 7.4 on 1940.11.10, 7.2 on 1977.03.04, 7.0 on 1986.08.30, while the strongest earthquake during last two centuries, with $M \geq 7.6$, occurred in 1802). The hazard analyses performed in INCERC led for the City of Bucharest to return periods in the range of 10 years for $I = 6.$, 20 years for $I = 7.$, 50 years for $I = 8.$, 200 years for $I = 9.$ etc..

Direct experience shows that, in Bucharest, the most endangered buildings are the relatively tall buildings built before 1940. They are handicapped by the lack of concern for earthquake resistant design, by low material quality, by the cumulative effects of the successive strong earthquakes referred to, by corrosion, fatigue due to urban traffic, sometimes also by unsuited interventions aimed to modify their functionality, finally by the fact that the predominant periods of ground motion tended to be close to their fundamental natural periods. Other categories of buildings are in general less vulnerable, but several of them do not fulfil general requirements and criteria, such as set by some modern codes. The post-war built repetitive tall buildings with r.c. structures, designed before 1977, raise such problems, even if only two of them underwent partial collapse in 1977. Another endangered category is that of relatively low-rise (up to 4-5 storeys tall) masonry buildings lacking r.c. horizontal diaphragms. An additional view on the situation of the building stock is provided by the outcome of a parametric probabilistic risk analysis. This study showed that, in case one leaves the most vulnerable buildings referred to previously as they are, for some decades to come, the collapse probabilities are in the range of several tens of percents. This is an unacceptable risk. The experience at hand shows the obstacles to intervention raised by the high costs and by the lack of a buffer area for occupants who should clear their apartments.

3. Considerations on the development of intervention strategies

The degrees of freedom of the solutions of intervention are related to structural, functional, architectural aspects. The number of degrees of freedom increases of course in cases one deals with urban areas. A degree of freedom not to be forgotten is the intervention time. Especially under conditions of frequently occurring strong earthquakes like those due to the Vrancea seismogenic zone, the time coordinate may play a crucial role. The alternative solutions or strategies may be characterized by several criteria. The problem of cost-benefit analyses under these conditions is obviously multi-criterial. In case one considers the different components of benefits and costs/losses, corresponding to different criteria, there are several factors, contributing with corresponding terms: benefits derived from normal service, costs of investment, costs of maintenance, costs related to the use, losses due to earthquake occurrence, benefits/costs related to the integration of buildings in urban systems and in plans of further development, perhaps others too. Cost-benefit analyses can be performed using different approaches and algorithms. One can look e.g. for a minimum of costs/losses or a maximum of net utility. Another approach is a differential one, where attention is paid to the additional cost per (expected) additional life saved. An approach to the cost-benefit analysis, proposed by the author, relies on a tabular format, where the various columns correspond to the different criteria, or non-commensurable components of benefits or costs/losses, while the various rows correspond to the different alternative strategies.
such cases it becomes interesting, if possible, to use non-Poissonian earthquake recurrence models, to account to some extent for the features of physics of earthquake generation.

4. Final considerations

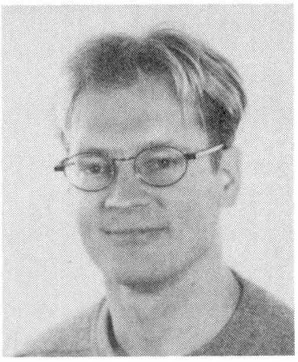
The importance of considering earthquake protection is in case one wants to save existing buildings located in seismic areas. On the other hand, the concern for earthquake protection should never lead to situations where other essential requirements set by now by the legislation in force in numerous countries are neglected when one develops proposals for decision related to the future of the existing buildings. The importance of developing a general strategy for partial preservation and partial, gradual, replacing of the existing building stock is obvious.

Earthquake Retrofit of Large Panel Buildings by Basement Isolation

Uwe E. DORKA
Head, Struct. Eng. Laboratory
University of Kaiserslautern
Kaiserslautern, Germany

Uwe E. Dorka, born 1956, has a Masters in Civil Engineering from the University of Washington, Seattle and a Doctorate from the Ruhr-Universität Bochum, Bochum. He researched structural control systems in the US, Japan, Austria and Germany. Since 1993, he heads the Structural Engineering Lab of the University of Kaiserslautern. He owns ***DSC*** which consults on structural control systems and manufactures control devices.

Esa FLYGARE
Research Engineer
University of Kaiserslautern
Kaiserslautern, Germany



Esa Flygare, born 1968, attained a Masters in Civil Engineering from the University of Oulu, Finland in 1993. He worked as structural engineer on several projects in Berlin, Germany. Since 1996, he is a research engineer at the Structural Division of the University of Kaiserslautern.

SUMMARY

Hysteretic device systems are known to provide very economical earthquake retrofitting solutions for non-ductile frames. Using this concept to retrofit large panel buildings, a “basement isolation” system emerges that can be implemented with very little disturbance of the tenants. This paper focuses on the structural realization and detailing of such systems. The problem of redistribution of vertical loads caused by the change in system is addressed and implementation details for the main system components, the seismic links and hysteretic devices, are discussed. It is concluded that this concept provides for very economical earthquake retrofitting solutions for LPBs.

1. Structural Concept of Hysteretic Device Systems

Hysteretic device systems (Dorka 1994) have three main structural features: A stiff primary horizontal load resisting system (PHS) with seismic links (SLs) and a soft secondary horizontal load resisting system (SHS) acting parallel to the PHS. The purpose of the PHS is to concentrate the horizontal deformations in the seismic links where hysteretic devices (Hydes) are placed. Such devices must be stiff and provide a stable elasto-plastic hysteresis loop. Thus, the horizontal forces in the PHS are limited to the maximum hysteretic device forces. Because of the large stiffness, the Hydes are activated at small displacements dissipating most of the earthquake's input energy (up to 85%). This in turn keeps overall horizontal displacements small. The SHS provides stability to the building during the mostly non-linear behavior of the PHS under severe earthquake loading. Well designed hysteretic device systems can have forces known only from very ductile systems and displacements known only from very stiff systems thus combining the advantages of both traditional structural approaches.

2. "Basement Isolation" for Retrofitting LPBs

Large panel prefabricated buildings (LPBs) are very stiff by nature. Introducing only one seismic link in the basement produces a hysteretic device system which can be appropriately termed a "basement isolation" system. For this, all transverse walls in the basement must be cut free so that all horizontal forces are transmitted through the remaining walls with the seismic links (fig 1). A reliability study of a 6 story basement isolated LPB based on 500 generated earthquakes (Dorka, Ji, & Dimova 1997) yielded the design curve for the device forces (fig 2).

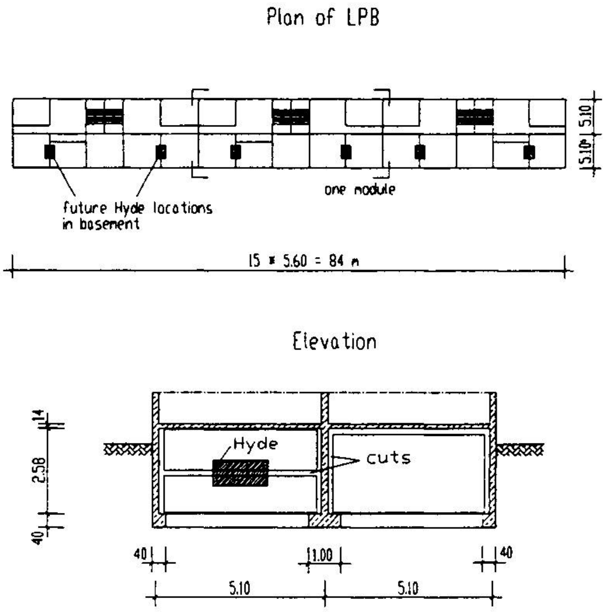


Fig1: Plan and basement elevation of a typical 6 story LPB with possible Hyde locations.

2. Design and Detailing of a Basement Isolated LPB

The most severe change to the existing system is the cutting or removal of the transverse basement walls. Ground floor walls and panel connections are in general able to redistribute the loads but the fundations usually need strengthening. The elastic rotational capacity of the basement wall connections defines the limit to the allowable drift and provides the design point in fig 2. Three devices may be used as Hydes that are simple to manufacture and implement: Shear panels, Honeycomb and friction plate. The least expensive should be the Honeycomb, followed by the friction plate. Shear panels cause the highest manufacturing and implementation effort of these three.

3. Conclusions

Basement isolation can dramatically enhance the performance of LPBs under earthquakes. Simple devices based on yielding or friction are available for the required performance range. Care must be taken to secure redistribution of vertical loads due to the change in system. Due to the limited interference in the existing structure and undisturbed usage of the building during retrofitting, this concept should be economically far superior over conventional retrofitting schemes.

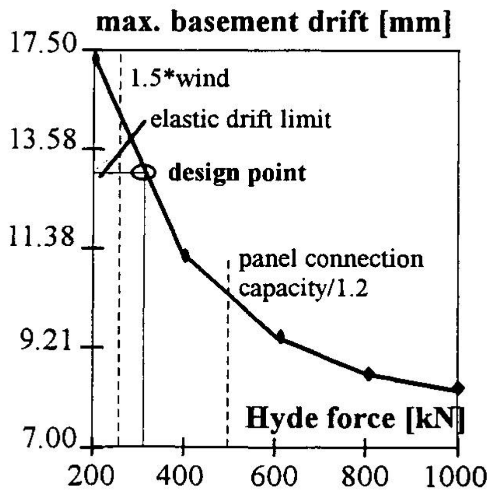


Fig 2: Design curve for Hyde forces in a 6 story LPB under Kalamata

3. References

Dorka, U.E. 1994. Hysteretic device systems for earthquake protection of buildings. *5th US NCEE*. Chicago. 1: 775-785.

Dorka, U.E.; Ji, A & Dimova, S. 1997. Earthquake safety of large panel buildings retrofitted with hysteretic devices. *ICOSSAR '97*. Kyoto.

Retrofitting of Reinforced Concrete Frame Buildings

Stephanos E. DRITSOS
Assistant Professor
University of Patras
Patras, Greece

Konstantinos G. VANDOROS
Civil Engineer
University of Patras
Patras, Greece

Colin A. TAYLOR
Reader
University of Bristol
Bristol, UK

Stephanos Dritsos, born 1951, is currently an Assistant Professor in the Department of Civil Engineering, University of Patras, specialising in the field of redesign of existing structures.

Konstantinos Vandoros, born 1970, is a PhD student in the Department of Civil Engineering, University of Patras.

Colin Taylor is a Reader in Earthquake Engineering in the Civil Engineering Department at Bristol and Manager of the Earthquake Engineering Research Centre there.

Summary

Shaking table tests that were conducted on a single-storey, small scale, 4-column, 3-D reinforced concrete frame model building retrofitted by concrete jacketing are presented. It is concluded that concrete jacketing is a very effective seismic strengthening technique for reinforced concrete frame buildings, since stiffness, strength and ductility can be significantly enhanced. Moreover an undesirable column-sidesway failure mechanism can be transferred to a desirable beam-sidesway one.

1. The building model

A series of shaking table tests were conducted on a reinforced concrete (RC) frame building model. The specimen was a single-storey, 4-column, 3-D specimen with a RC slab at the top and one bay in each direction, simulating a real structure at 1:4 scale. The external dimensions of the plan of the model was 810mm by 1460mm and the height of the columns over the foundation beams was 640mm. The cross-section of the beams and the columns was 80mm x 80mm.

The retrofitting procedure involved the construction of RC jackets around the four columns. The thickness of the jackets was 25mm and they were cast from the foundation level up to the top of the column. The surfaces of the columns, were roughened by a pneumatic scrabbler prior the application of the jacket. The maximum aggregate size was 10mm. Jacket reinforcement consisted of four 6mm diameter longitudinal bars and 3mm diameter hoops fixed to the main bars at a spacing of 20mm. The hoops consisted of two L-shaped portions with 45° end hooks which overlapped in diagonally opposite corners which were alternated at each layer. The longitudinal bars were anchored into 60mm-deep holes which were drilled in the foundation beams and subsequently filled with epoxy adhesive. The other end of the jacket bars was welded on special steel plates already placed in the beam-columns joints of the initial model. No jacket hoops were provided in the beam-columns joints to avoid damaging the beams.

2. Shaking table tests

In the present work, shaking table motions were restricted along the longitudinal axis of the specimen. Since, simple sine motions ease the identification of model failure, sine dwell input motions with constant frequency were used. Thus, only the acceleration level and the frequency of the sine wave was changed at each test. It was desirable to shake the specimen at a frequency near to resonance. Therefore, the testing frequency was chosen about 10% lower

than the natural frequency of the model since a frequency reduction was expected due to damage during testing. Moreover, a set of low level exploratory tests was conducted at the beginning of testing and after the main seismic tests to characterize the natural frequencies, mode shapes and damping parameters of the model. Accelerations, displacements and steel strains were measured at various locations.
Seismic tests were performed on the unretrofitted model with an additional load of 10 kN on the top of the specimen to increase the induced inertia forces. Testing was stopped just one step before total collapse of the specimen with severe damages at the top and bottom of the columns. The initial natural frequency of the specimen in the longitudinal axis was found to be 26.7 Hz. However, the natural frequency dropped to 8.7 Hz when a kentledge of 10 kN was positioned on the top of the model. Failure of the model occurred in a peak table acceleration of 2.5g, and the last seismic test was interrupted to prevent total collapse. Plastic hinges created in the columns resulted in the development of a column-sidesway failure mechanism as was anticipated. This, final shake was performed using a sine dwell input motion of 6.0 Hz. The natural frequency before the test was 6.75 Hz, and it dropped to 4.86 Hz after testing. In Fig. 1, acceleration time histories on the table and on the roof of the model are shown. It is clear that damage to the model initiated very early.

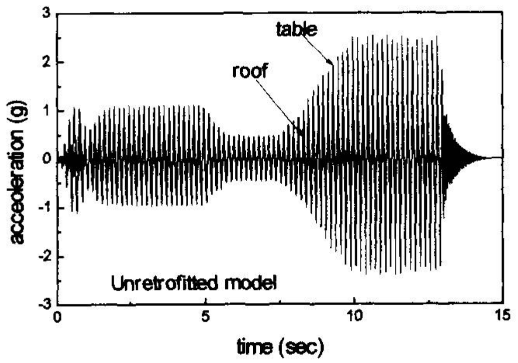


Fig. 1 Acceleration time histories on the table and on the roof of the unretrofitted model

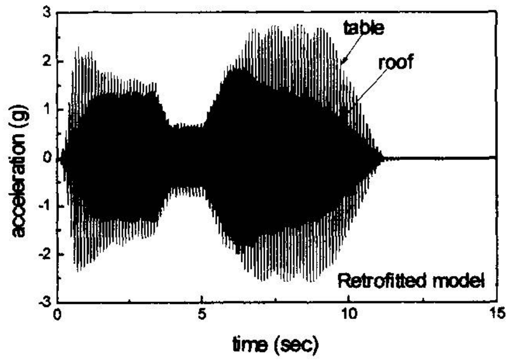


Fig. 2 Acceleration time histories on the table and on the roof of the retrofitteed model

The natural frequency in the longitudinal axis of the retrofitted specimen was found to be 47.3 Hz which dropped to 11.75 Hz when a kentledge of 10 kN was added at the top of the specimen. Eleven seismic tests were performed with a kentledge of 10 kN at the top in a range of 0.50g to 2.60g peak table accelerations. In Fig. 2 the acceleration time histories on the table and on the roof of the model are shown for the test with a peak table acceleration 2.60g where plastic hinges started to form at the bottom of the columns and beam ends. At the end of testing the natural frequency was measured to be 5.45 Hz. However, the natural frequency dropped to 4.25 Hz when the additional weight on the top increased from 10 kN to 30 kN. Finally, four more seismic tests were performed with a kentledge of 30 kN at the top up to a shaking table acceleration 1.65g. From the test series it was evident that the concrete jacket contributed significantly to the seismic resistance of the structure. High levels of strains were measured on the longitudinal reinforcement of the jackets while the strains were considerably lower in the respective bars of the original columns. Finally it is concluded that concrete jacketing is a very effective seismic strengthening technique for reinforced concrete frame buildings, since stiffness, strength and ductility can be significantly enhanced. Moreover an undesirable column-sidesway failure mechanism can be transferred to a desirable beam-sidesway one.

Damage of Reinforced Concrete Structures Exposed to Violent Thermal Gradient

Paolo CIONI
Assistant Professor
Univ. degli Studi di Pisa
Pisa, Italy

Pietro CROCE
Assistant Professor
Univ. degli Studi di Pisa
Pisa, Italy

Walter SALVATORE
Researcher
Univ. degli Studi di Pisa
Pisa, Italy

Summary

The evaluation of the actual damage suffered by reinforced concrete buildings exposed to fire is necessary to decide if rebuilding is more advantageous than repair or vice versa. An original assessment technique is proposed, in which thermal and stress theoretical analyses are combined together to study a posteriori fire damaged structures, being the maximum temperature attained by the actual fire deduced by searching, using spectroscopy, temperature dependent mineralogical transformations of basic components of the concrete aggregate. A worked example, concerning a fire damaged existing industrial building, demonstrates the efficacy of the method.

1. Introduction

The refurbishment of reinforced concrete buildings exposed to fire is a problem of great topicality. In fact, since in many cases the buildings are not significantly destroyed, it is necessary to decide when rebuilding is more advantageous than repair or vice versa. In making such a decision, the evaluation of the actual damage suffered by each structural element becomes crucial, in order to distinguish the structural parts to pull down from those that can be repaired. Unfortunately, fire injuries are often not restricted to the external surfaces of beams and columns and penetrate deeply into their core, so that appropriate diagnostic procedures, combining non-destructive testing methods with theoretical thermal and structural analyses, are required. Moreover, non destructive testing methods result not yet satisfactory, and therefore, at present, it seems quite impossible to estimate exactly inner damages, unless a posteriori exhaustive information, like the knowledge of the maximum temperature attained on the surface of each structural element, to be deduced by multidisciplinary analyses, is available. On these bases an original method for the assessment of existing building subjected to fire has been developed.

2. A refined combined method for damage diagnosis

The procedure consists in the preliminary localisation by non destructive testing methods of the most damaged areas, to be analysed in most refined way. Subsequently, analysing cores taken out from the elements themselves, the internal crack patterns is mapped while the maximum fire temperature is deduced, resorting to spectroscopy, checking the penetration depth of characteristic mineralogical transformations, affecting certain mineral components of the aggregate, subjected to high temperatures, like the one concerning dolomite, which originates, at 832 °C, brucite and periclase. In this way the maximum temperature attained on the external surface during the fire can be evaluated, allowing the calibration of the input fire curve for transient thermal and stress FEM analyses.

3. Diagnosis of existing structures exposed to fire: a worked example

The validity of the procedure sketched out before has been proved studying the damage of an existing industrial building, a paper-mill located near Lucca (I), which was seriously injured by fire in the spring 1997.
In fact, the coring of the columns permitted to stress the internal crack pattern of the damaged column, characterised by cracks propagating perpendicularly to the exposed face, while the spectroscopic analysis has proved that the transformation of dolomite in brucite and periclase affects a 6 mm thick layer of concrete, so that the FEM transient thermal analysis of the reinforced concrete column shown that the maximum temperature on the heated face of the column during the fire was about 957 °C.
The results of the stress analysis show that tensile stresses parallel to exposed surface occur, which are much higher than the tensile strength of concrete, and this explains the reason of the opening of the detected cracks.

4. Conclusions

An original procedure to assess fire damaged existing reinforced concrete buildings, making use of multidisciplinary knowledge, has been developed. The method is based on preliminary non destructive tests, mainly ultrasonic, which allow to establish the most damaged elements on which further investigations must be focused. Beside that spectroscopic analyses are carried out in order to discover, studying suitable mineralogical transformation of the aggregate components, some characteristic temperature, that will be used to locate the position, inside the element, of the corresponding isotherm. This information, by means of an appropriate theoretical analysis, leads to fix the maximum temperature attained on the exposed surface during the fire, in such a way that the appropriate input data for thermal and stress analysis of the building can be set up.
A worked example, concerning an industrial building severely injured by fire, is fully developed, demonstrating the flexibility and the powerfulness of the proposed method, also explaining the complex crack pattern discovered in the damaged columns.
The development of the method, still in progress, is mainly addressed toward the improvement of non destructive investigations to calibrate the input data.

Repair and Rehabilitation - Three Case Studies in Bucharest

Carmen BUCUR
Reader
University of Bucharest
Bucharest, Romania

Carmen Bucur, born 1951, received her civil engineering degree from Bucharest University in 1974 and PhD in 1994. She is Reader - Department of Structural Mechanics, Technical University of Civil Engineering Bucharest, Romania

Neculai CHIVU
Structural Engineer
University of Bucharest
Bucharest, Romania

Neculai Chivu, born 1938, received his civil engineering degree from Bucharest University in 1961. He is a consultant, Certified supervisor and Quality assurance (AQ), with Design Institute of Railway Bucharest, Romania

Aurel ARDELEA
Structural Engineer
University of Bucharest
Bucharest, Romania

Aurel Ardelea, born 1940, received his civil engineering degree from Bucharest University in 1963. He is a consultant, Certified supervisor, Head of Structural Department with Design Institute of Railway Bucharest, Romania

Summary

Strengthening solutions of three buildings in Bucharest are presented. Two buildings are part of the Technical University of Civil Engineering and the third is a warehouse for high value equipment. They have been affected by three major earthquakes: in 1977, 1986 and 1990. The strengthening solutions were adopted after an accurate analysis on seismic structural performances. The three buildings have different static and spatial shape. The structural models are spatial, with uni- and bidimensional finite elements. The structural analysis was performed in elastic range with a common programme.

Keywords: Building , Seismic Assessment, Rehabilitation, Earthquake, Technical regulations.

Introduction

Bucharest, the capital of Romania, is affected by the earthquakes originated in Vrancea Region (Fig.1). These earthquakes have two main characteristics: the persistence of their focus in almost the same place and the hypocenters depth between 60 and 300 km [2].

The aim of the study was to locate the vulnerable points of the structures and their necessary rehabilitation methods. All the tree structures studied with present paper – reinforced concrete frames with rigid floors - have been spatial modelled with finite unidimensional finite elements (the frames) and bidimentional elements (the floors).
Each structure has been calculated under as many modes of vibrations were required for the modal mass to be summed up in the closest ratio to 100 %. The structures spectrum response were determined based on Code design spectra. The damping was considered to be 5 %. The structures were successively loaded on two perpendicular directions in their plan. There have been chosen the less favourable response from the stress requirements point of view.

Fig. 1

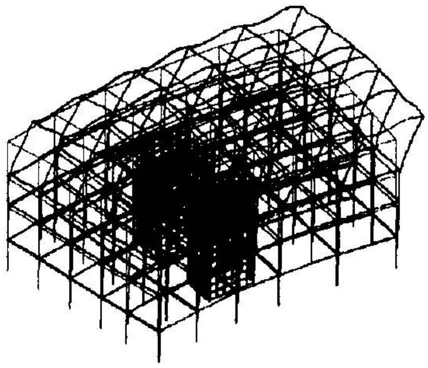


Fig. 2

"ROMTELECOM warehouse"

The building was built in 1940. It has a basement, a ground floor and two storeys with different heights (3.7 m, 4.15 m, 3.6 m, 4.55 m). In plan it has the shape of a circular sector with ca. 80.0 m radius and total area is 6180 m^2. An expansion joint has been provide.
The magnitude of the interstory drifts are of ca. 3.5 times larger than those required by the Code. The most stressed columns are: for Section A those in the corners of the buildings; for section B those of the outside bay. The structure has no redundant reserves and requires strengthening

Proposed solution for the rehabilitation is the same for both sections aiming to reduce the interstory deflection to that accepted by the Code. The solution consist of newly-added structural reinforced concrete walls interacting with existing frames. The displacements and the moments were greatly reduced in the existing structure. The maximum interstory drift is 0.0073 and is almost as with that required by the Code (0.0070). The sum of maximum displacements of the two sections is less than the size of the expansion/contraction joint. Fig. 2 presents the deformated shape of the strengthened Section B.

"The Buildings of the Faculty: Railway Department"

The building of the Railway Department - old part of the building - was built in 1947-1948. It is "L" shaped in plan. The main section has basement, ground-floor and three floors. The secondary section, was initially built with a high semi-basement (where is the sport hall) and one story, and between 1963-1965 two other additional storeys were built. The new building was built during 1972-1973. It is next to the old building previously described, separated by it through a joint, the two building allowing a free traffic flow between them. It is developed over six levels (basement, ground floor, four full floors and a partial one) and has a reinforced concrete elevator cage.

The buildings behaved reasonable well during the 1977, 1986 and 1990 earthquakes, with only small cracks in fill-mansory and local damages in the zone adjacent to the joining.
The buildings have structural redundant reserves allowing them to take over further seismic shocks. The building were proposed to be preserved without major strengthening works.
If some architectural works are to be undertaken a close qualified supervision must be assured to locate any hidden structural damage. The corresponding strengthening solutions must be provided and implemented if required. Also the joint between the two buildings must be cleaned.

Analysis of Deterioration of Civil Buildings in South Ukraine

Andrey MARKOV
Dr., Assistant professor
Zaporozhye State Engineering Academy
Zaporozhye, Ukraine

Pavel KOKOSHUEV
Post-graduate
Zaporozhye State Engineering Academy
Zaporozhye, Ukraine

Maria MARKOVA
Post-graduate
Zaporozhye State Engineering Academy
Zaporozhye, Ukraine

Summary

The southern regions of Ukraine are known for particularly numerous cases of deformation of structures built on collapsing soils. Deformation of different building types and various method of containing it have been analysis as well as the cost of damage repairs determined. The stress-strain state of buildings with foundations in the condition of inhomogeneous deformation has been defined by applying the Finite Element Method.

1. Protection of buildings against collapsing deformation

The following protective measures are usually carried out in the standard practice of public and residential building construction:

- driving of precast concrete piles through the entire collapsing soil layer;
- eliminating the collapsing properties of the soil by compacting its entire layer;
- applying a package of designing waterproofing measures by way of setting up two-metre-thick pad foundations, concrete girdles, contraction joints, etc.

The most expensive but at same time the most reliable is the first of the above procedures, while the third one is the least expensive.

A protective procedure is to be selected by taking into consideration both the soil and the type of a building. Single houses are built without any special protection. Buildings of up to six stories are protected mainly by a package of design and waterproofing measures. About 50% of 7-10-storey buildings are also protected by the same package when 45 percent of them are erected on compacted soil. And only 5% of them are erected on pile foundations. Majority of 10-storey or taller buildings are put up on pile foundations and only some of them—on compacted soils.

2. Deterioration of buildings

Buildings in service are deteriorated because of inhomogeneous moistening of the underlying soil mainly due to water leakage out of water conduits. The kind of deterioration generally depend on the type of building. The most serious deterioration which are sustained by buildings of four to six stories are due to their hogging. Repair expenditures comprise 8-10% of the building cost. The observation data on the behaviour of deteriorated building have been statistically analysed and the probability of building trouble free service during their rated lifetime has been determined.

3.Determination of stress-strain state of a building

Calculations based on the revised models were used in order to substantiate the recommendations of design and protective measures for buildings both being deformed and under constructing.
The "building-collapsing soil" system is approximated:

- by solid elements simulating soil with natural moisture content, the moistened soil tract and the zone of compaction;
- by beam elements simulating the piles and the elements of building strengthening;
- by shell elements simulating the building structures.

Using such models facilitates the evaluation of the stress-strain state caused by building deformation. The calculated stress-strain state corresponds to the in-situ damage measurements thus testifying to the adequacy of adopted computation models. Besides the actual influence of settlement upon the building in way of local reduction of the foundation rigidity caused by moistening is to be taken into consideration
The investigations thus carried out, and the analysis of the building stress-strain state in particular, permit of increasing the building reliability by the efficient designing, constructive measures and strengthening the deformed buildings.
Several methods can be used to further the service of deformed buildings:

- strengthening the foundations or arrangement of additional pile foundations using pressure piles;
- strengthening the building structure by steel elements including those prestressed;
- eliminating the building leaning by soil removal from under the foundation;
- jacking up the building;
- induced moistening of collapsing soil.

These are comparatively labour consuming and expensive operations, not always performing their mission. Methods of selecting reinforcements based on calculation promote the reliability of buildings and reduce the expenditures.

Antiseismic Protection and Rehabilitation of Buildings in Timisoara

Decebal ANASTASESCU
Prof. Dr. Eng.
ILIAN - Ltd
Timisoara , Romania

Angelu GADEA
Prof.
CASTEL - SD - Ltd
Timisoara, Romania

Decebal Anastasescu, born in 1926, received his civil engineering degree in 1951 and PhD in 1973. He has worked at the "Politehnica" University and in various design institutes of Timisoara. His main research is in the field of the reinforced concrete space structures - foundation soil interaction.

Angelu Gadea, born in 1925, received his civil engineering degree in 1949. He has worked at the "Politehnica" University and in various design institutes of Timisoara. His main research fields are the geotechnical investigations and soil consolidations.

Summary

In the paper are classified the existing buildings of the town, from the point of view of vulnerability, in the case of a major future earthquake. Varied repair and rehabilitation methods are briefly described, applied especially to some historical or architectural monuments, according to actual antiseismic national codes, in correlation with the preoccupation for characteristic architectural conservation, new functional demands and investment funds. Finally, the authors propose some necessary measures, with a view to achieve a high technical and economical efficiency level of the antiseismic protection in Timisoara.

1. Introduction

Timisoara is situated in the Plain of Banat, well - known for a seismic medium activity (maximum 8 degree on the MM scale), with deepness focus of 4 to 10 km epicentral zones reduced as area. The soil stratification of alluvional nature contains, at the surface, the complex of difficult soils, made of heterogeneous fillings, silt ground and loose sands. Underground water rises just a reduced deep of 2 to 4m.

2. The vulnerability of existing buildings

The ensurance degree for a future major seismic action, of existing buildings in Timisoara, presents great differences due to both code requirements evolution, applied to these buildings construction and the official application of these codes and also due to the effects of " time factor" (chemical aggresivity, uneven foundation settlements, overloads from former earthquakes or functional modifications).

Therefore, the highest vulnerability it is noticed to old buildings, that have been built before the first world war, many of these buildings have a well - known architectural and historical value. Nor existing that epoch any codes in the field of antiseismic design, these constructions have no correct antiseismic conformation (structural system), nor strength capacity, or necessary ductility of the structural components, setting also some deficiencies concerning construction, completed by the unfavourable effects of "time factor". The absence of the antiseismic protection measures have unfavoured also the buildings constructed during the period between the two world wars. Since 1977, Timisoara, included officially into seismic zone of 7 degree intensity, has benefited from code requirements (P 100 - 78, P 100-81), and since 1992 its degree has been increased from 7 to 7.5, keeping with P 100-92 code.

3. Existing structures consolidation

The problem of antiseismic protection of existing buildings in Timisoara, although it has been revealed some 15 - 20 years before, has been seldom tackled (on the occasion of some functional transformations or elimination of some damages due to surrounding medium), because of the little funds. At present, the importance of this problem rises because of:
- the recent increase of the seismic intensity degree;
- the amplification of functional modifications (horizontal or vertical extensions) due to the constructional space crisis and proliferation of economic units, in the difficult conditions of transitional period of our country.

Among the old buildings, in the perimeter of United Square (realized between 1730 - 1750) included in the historical zone of town, submitted to some transformations and consolidations, one mentions especially: Constructim Residence, The Baroque Palace (The Art Museum today), Bankoop Residence.

Not having the necessary funds, it has been proposed, in some cases, the structural consolidation on steps: after foundation soil or foundation consolidation and of cracked structural elements ones, there is provided, in the next stages, the introduction of supplementary shear walls, columns, belts, tie - pieces, anchoring and increase of in plane rigidity of timber floors (through orthogonal disposal of metal beams) or the replacement of timber floors with reinforced concrete ones. In this way there were realized some consolidations at: The Huniade Castle (The Banat's Museum lately), The Building of Romanian Opera, The Church of the Piarist High - School, The Building of Romano - Catholies Bishopric.

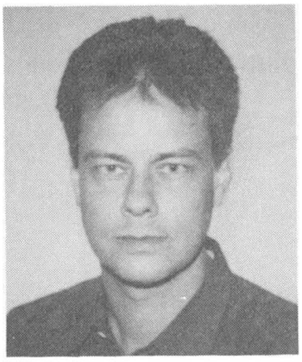
Among the extension works that have been realized lately, there are mentioned especially: The Baby care Center Complex, The Orphan children Boarding School.

4. Conclusions

In order to realize the antiseismic protection, at a high level of technical and economical efficiency, of the existing buildings in Timisoara, the authors consider some necessary measures, starting with the determination of the ensurance degree for seismic actions of these buildings and ending with the performance of the consolidation works by specialized enterprises, able to assure the standard prescribed in the construction projects. The achievement of the above objectives needs the tight cooperation of all factors involved in this action (research, design, construction, local and central administration), the general level of antiseismic ensurance of existing buildings and the urgent application of these necessary measures being dependent of economical possibilities of investors, of entire society.

Emergency Protection of War Damaged Buildings

Niels STRUFE
Civil Eng
DEMEX Consult. Eng.
Copenhagen, Denmark



Niels Strufe, born 1963, received his engineering degree at the Copenhagen Technical Institute in 1990. He is presently Head of Section with responsibility for Disaster Recovery and Demilitarisation operations.

Summary

The Protection of War Damaged Buildings and Waste Management, Reconstruction of Mostar, Bosnia and Herzegovina, was performed for the European Administration of Mostar (EUAM) following the local cease-fire agreement in 1994 between the Muslim and Croat warring fractions. The *protection of buildings* comprised all kinds of partial demolition and construction work in order to protect buildings against further destruction and to protect people against the risk of structural collapse and falling objects.

1. Scope of Protection Work

Generally, the protection work comprises all "protected buildings" (historical buildings) exposed to damage excluding those which must be demolished due to the extent of damage. However, a number of other important buildings also required protection.

The *protection work* was planned and conducted according to engineering design of the individual buildings with respect to future plans of repair and reconstruction. The engineering designs are based on a detailed survey of the structural stability and risk analyses of structural failures and collapses, including the risk of seismic impact (level 6 area). In some cases the protection work was performed as *emergency demolition/protection work* comprising the most urgent precautions to secure the building and the public against any hazards.

For the whole of the City of Mostar, it is expected that 30% of Damage Category 5 (DC5) buildings and 90% of Damage Category 6 might be demolished which results in a planning figures of totally 1000 buildings with130,000 m^2 GFA for demolition and after evaluating the individual options for demolition or reconstruction comprises a total gross floor area of approximately 200,000 m^2, the amount of demolition waste roughly amounts to 200,000 tonnes. To this amount should be added wastes arising from the reconstruction works of buildings and infrastructure.

2. Implementation

The EUAM and the Municipality of Mostar agreed that the physical condition, historical value and ambient situation should form the priority of demolition. Referring to the EUAM Decree on

Demolition of Building Structures, 19 March 1995, the EUAM would provide for the demolition (partial and total) of damaged buildings which

- endanger the lives and health of people or property, or
- hinder efficient use of communications or other structures, or
- disrupt the sight of the immediate environment to a considerable extent, or
- stand in the way of construction of dwellings, industrial or other structures.

Some of the most important buildings of historic and cultural interest have been surveyed together with UNESCO and the importance of the buildings and the protective measures have been discussed and agreed upon.


Fig. 1 & 2 Example on Emergency Protection of building with historical value.

3. Conclusions

The results of the work are very satisfactory. It has been a very positive experience for the local citizens and to UNESCO to save so many historical buildings from complete destruction. The buildings are now protected, ready for reconstruction and a number of buildings have already been repaired and some are under reconstruction. Certain specific problems have been encountered during the protection and demolition works, these including

- presence of unexploded ordnance (UXO) which must be cleared by authorised personnel
- location of disposal sites for reusable materials and appropriate disposal sites
- time consumption for appropriate discussion & approval by local Municipal Administration
- lack of local equipment and experience.

Based on the experiences of the presented emergency protection and demolition work in Mostar it is evident, that there is an urgent need for the concept of Emergency Building & Solid Waste Management integrated with post war Rehabilitation Programmes.

Protection of Church Buildings in Mining Areas

Marian KAWULOK
Head of the Institute's Division
Building Research Institute
Gliwice, Poland

Marian Kawulok, born 1941, received his civil engineering degree from Silesian Technical University in 1964 and PhD in 1980. He is currently head of Division of Building Research Institute in Gliwice and lecturing on Silesian Technical University.

Summary

Church buildings erected as traditional structure arrangements with brick walls and shell vaults are characterised by low spatial stiffness and low resistance to tensile forces. Therefore, they are especially sensitive to all ground movements, including land deformations caused by underground mining activity. The protection of church buildings is an important technical and social -economic problem in the Silesian Mine Basin in the south of Poland, where intensive underground mining is still conducted on large scale. Generals rules of the assessment of mining activity on church buildings and structural measures of their protection are discussed in the paper. Examples of strengthening and other solutions considering the protection of the existing buildings subjected to mining influence are presented, together with a short analysis of their efficiency.

Key words: mining areas, church buildings, mining trough, structural measures of protection

1. Impact of mine-induced subsidence trough on buildings

Land deformations which result from underground excavation activities, are manifested on the surface mostly in the form of regular subsidence trough (Fig. 1).

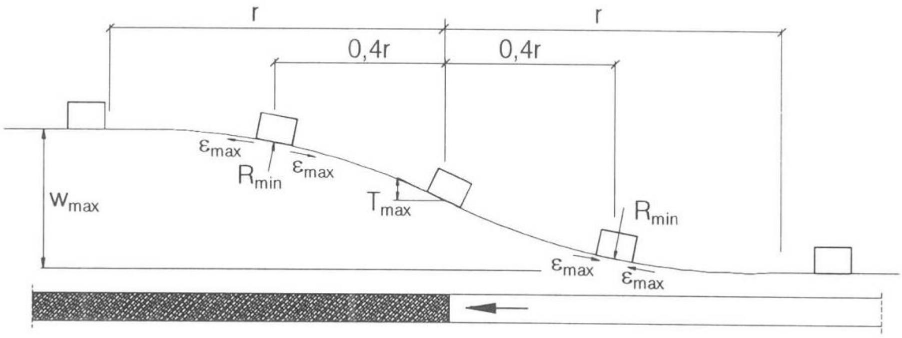


Fig.1 Typical layout of a building situated in mine-induced subsidence trough, r- radius of mining impact range.

The shape of the subsidence trough may be described by the following indices, whose rate is determined by the geological and mining conditions pervading in given areas [1]: vertical displacement (subsidence) w, horizontal displacement u, tilt T, curvature K (or curvature radius $R = 1/K$) and horizontal strains ε.

2. Principles of church buildings protection

The diagram of the impact of convex subsidence trough upon the diagonal bearing elements of the discussed structures is illustrated in Fig. 2. Following the horizontal displacement of the foundations of pillars and walls due to the impact of horizontal strains, the supports at the vault abutments level are also displaced (Fig. 2 a). The impact of curvature radius evokes analogous results as to the quality of structure condition. Following the rotation of the supports of the structure, its abutments may also be horizontally displaced (Fig. 2 b). In both cases, the values of initial compressive stresses are reduced in all structural elements, which poses particular hazards to arches and vaults. Nevertheless, the most disadvantageous effect is produced when the building is set diagonally in the subsidence trough (Fig. 2 c), because its corners subside at different rates ($w_1 \neq w_2 \neq w_3 \neq w_4$). Due to low spatial stiffness characterising traditionally constructed church buildings, such impact results in considerable hazards to the resistance and stability of church vaults.

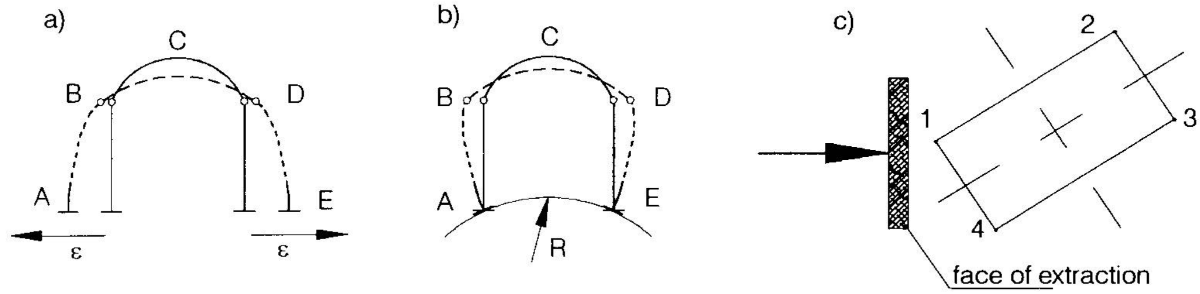


Fig. 2 Diagrams of the impact of subsidence trough on buildings.

The rudiments of protecting the bearing elements of church buildings located in mine - induced subsidence trough involve the limitation or elimination of the displacement occurring at the foundations level A-E , as well as at the level of vault abutments B-D (Fig. 2 a, b). On principle, if the foundations are strengthened, the detrimental impact of horizontal strains on the whole bearing structure of a building is eliminated. On the other hand, if the structure is braced at the B-D level, the impact of ground deformations on the structural elements of the vaults, which are particularly vulnerable, is theoretically excluded. However, it should be emphasised that to reduce the detrimental impact of the mining works conducted diagonally, the strengthening elements should have a truss structure, as a rigid system in the plane. This is particularly important to the bracing at the vault abutment level, plane B-D. Hogging or sagging ground curvature evokes uneven subsidence of the building walls. It is only possible to protect the walls against bending by bracing at the vault abutment level, or, if such strengthening is feasible, at the intermediate levels.

The paper presents examples refer to the protecton of three aisle church buildings, whose bearing structures consist of the exterior brick walls and interior system of pillars, separating a higher central nave. Brick arches are supported by the walls and pillars. The space between the arches is filled with gypsum shell in the form of a vault. In plane view, these buildings have the shape of a cross, with different dimensions. They also differ in their bodily shape, structural solutions and the geological and mining conditions of their foundations.

Given examples allow to say that preventive treatment of church buildings in mining areas includes the methods which usually provide for the safe usability of churches, but do not exclude the possibility of the occurrence of even major damages, as such structures are characterised by low resistance to mine- induced ground deformations.

Reinforcement of Apartment Buildings Exposed to Paraseismic Tremors

Jeremi M. SIECZKOWSKI
Professor
Techn. Univ. of Wroclaw
Wroclaw, Poland

Grzegorz DMOCHOWSKI
Dr. Sc
Techn. Univ. of Wroclaw
Wroclaw, Poland

Czeslaw BIELAWSKI
Dr. Sc
Agricultural Univ. of Wroclaw
Wroclaw, Poland

Summary

A method of reinforcing prefabricated ferroconcrete apartment buildings subjected to mining earth tremors has been presented. The reinforcement consists in the extra bracing of the longitudinal walls of the staircases by making them thicker and by adding external pillars.

1. Introduction

In the 60s and 70s many new housing developments were built in the towns of Glogów, Legnica and Polkowic (the conurbation of these towns is referred to as LGOM). This was associated with the beginning of the mining of the copper ore deposits there. Since initially it was assumed that the area of mining would not reach the towns, the structures that were erected there were not protected against the effects of mining operations. Soon, however, copper ore was mined not only close to these towns but even under them. This resulted in damage to several structures previously erected in this area. This happened, for example, in Glogów where it became necessary to reinforce five apartment buildings because of their considerable damage and excessive deformation.

2. Description of Reinforced Buildings

The considered structures are located in the *Copernicus* housing development. These are isolated, 13-storied buildings situated on a 11×46 m rectangle plan. They were built from prefabricated reinforced concrete wall plates and floor slabs. The transverse walls perform the load-bearing function. The external walls were constructed as curtain walls welded to the transverse walls. The walls of the staircases longitudinally

constitute the bracing. A scheme of the structural system of one of the buildings is shown in Fig. 1. The buildings are founded on foundation plates.
According to the construction system inventors' design, the vertical and horizontal joints between the prefabricated wall and floor units were to be packed tightly with concrete or, in places, with cement mortar. In reality, because of shrinkage and temperature phenomena, natural deformation, and movements of the building as well as sloppy execution, the concrete and the mortar in the joints soon started cracking and spalling. As a result the spatial rigidity of the building decreased markedly.

3. Paraseismic Phenomena in LGOM Area

The genesis of a mining earth tremor and the mechanism of its action on buildings have been described in a number of papers [1, 2, 3, 4, 5]. These effects are commonly described by the following parameters:

- terrain surface subsidence,
- terrain surface inclination,
- terrain horizontal unit strain (terrain creep),
- the radius of the trough,
- the acceleration of the horizontal vibration of the subsoil.

According to the forecasted effects of the projected mining [7], the first four of the above quantities will be small enough not to cause the destruction of the considered buildings, particularly that they are founded on foundation plates. Whereas the expected horizontal acceleration of the subsoil producing kinematic excitations in these structures is great enough to cause their failure.
According to [6], the boundary value above which inertial forces should be taken into account in the design is the terrain horizontal acceleration of 50 mm/s^2. The maximum acceleration measured so far in the LGOM area is 291 mm/s^2 and the boundary forecasted acceleration should not exceed 400 mm/s^2.

4. Damage to Considered Buildings

The following kinds of damage caused by mining operations were found in the considered buildings:

- cracks in the joints between the wall prefabricated units and the shielding prefabricated units. The cracks were 0.5 ÷ 0.2 mm wide;
- the separation of the curtain walls from the transverse load-bearing walls. The gaps were a 0.5 ÷ 0.5 mm wide.

The tenants complained that during tremors they could feel the building moving, the lamps were swinging widely, objects were falling from the shelves, so that they felt the urge to flee from their apartments. The subjective impressions were confirmed by measurements which indicated that the walls displaced horizontally up to about 70 mm.

Structural Assessment and Strengthening of Existing Buildings

Alexandru CATARIG
Professor
Technical University
Cluj-Napoca, Romania

Ludovic KOPENETZ
Professor
Technical University
Cluj-Napoca, Romania

Pavel ALEXA
Professor
Technical University
Cluj-Napoca, Romania

Summary

The existing concrete structures, through cumulative effects (earthquakes, corrosion, etc.) diminishes their bearing capacity and, therefore, their safety. On the other hand, the bearing capacity of the initial structure is strongly influenced by the state of knowledge included in the codes that govern the design. Thus, the aseismic design code P13-63 (valid in Romania in the sixties and seventies), that governed the seismic design for about 20 years, underestimated the design seismic loads, leading to structures with an initial low level of safety. If for structures designed according to aseismic codes, the safety level can be reasonably assessed, the assessment of safety level of structures designed and built before such codes have been enforced, is much more difficult. The study presents practical procedures to asses the safety level and strengthening techniques aiming at increasing this level and retrofitting the existing R/C structures.

1. Introduction

The built heritage has to be protected and preserved, in its quality of representing the society, and having to be passed to the next generation. The building component of the human activity reflects, among other things, the level of development of the society concerning the technical and economical regulations. The preserving measures of the built heritage has to be based on pertinent studies regarding the effects of negative factors (earthquakes, corrosion, etc.) and their cumulative action. The repairing, rehabilitation and retrofitting measures depend heavily of this study. The retrofitting of damaged buildings (updating the building to the current provisions of the technical codes) has to be the objective of the preserving activity. From the structural view point, some of the principal mechanical characteristics are considered in the retrofitting activity such as: strenght, stiffness, ductility and structural redundancy aiming at avoiding the whole possible failure of the structure. In this respect, the paper presents several aspects related to the possibility of increasing the safety level of existing R/C structures by way of affecting both, their infra - as well as their superstructure.

2. Fundamental concepts

The fundamental concepts that have to make up the basis of increasing the safety level are:
- the increase of local strength such that all the critical zones of the structure reach a strength level that mach the code requirements,
- avoiding any failure due to shear force of the structural elements and of the beam - column connections,
- avoiding any plastic behaviour of the foundations,
- avoiding the concentration of plastic hinges at one level only,
- ensuring a high plastic rotational capacity for all potential plastic hinges.

For the existing R/C structures (either skeletal or shear - wall structures), the most difficult problem in retrofitting consist in assessing:
- the critical zones, i. e. the zones where the plastic hinges may take place,
- the residual strength, i. e. the strength of the structural elements after they underwent cyclic deformations, due to mainly seismic actions but also after damages caused by corrosion, improper use, etc.

Strengthening of the foundation is a complex activity that requires the investigation of the structure, the foundation and the soil. A common feature of the proposed solutions is the altering of old structural statical model by introducing new structural elements of the type of elastic supports. The change of the mechanical model, as strengthening solution aims at limiting the actions upon the foundations to the available bearing capacity.

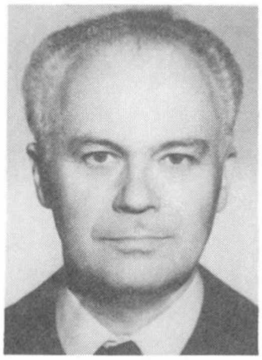
3. Strengthening of R/C structures

If the strengthening of R/C columns is achieved easily in most of the cases by applying a new R/C layer around the column, in the case of beams a careful analysis is required in order to select the optimal procedure. For instance, in the case of a frame structure, by altering the stiffness of the beam in the supporting zones, the middle span bending moment decreases by half and the displacement by 70 %. In this way, it has been avoided the strengthening of the beam along its entire span.

In Romania the number of buildings with precast large - panel structure is very high in the existing inventory. Therefore, the problem of increasing their safety level is vital and currently focused by Governmental agencies. During the repeated seismic activity in the period of 1977 - 1990, though no damages due to shear force have been encountered, the cracking of the poured in zones facilitated the corrosion of the reinforcement, which in its turn strongly affects the strength and stability of these structures. Solutions of increasing the safety level in such cases may be:
- doubling the cracked shear - walls, solution that can be applied in any situation but which requires long time and high costs,
- providing external columns with variable sections in the axes of existing shear walls. The columns are linked to each other by inclined or horizontal tensioned elements placed in narrow flutes made in the shear - walls.

A Case Study on a Historical Monument and Methodological Implications

Horea SANDI
Research Advisor
INCERC
Bucharest, Romania



Olga STANCU
Research engineer
INCERC
Bucharest, Romania

Summary

References are made to a French - Romanian project on the rehabilitation of a XVII - th century monastery, representing a historical monument. The analysis presented in the paper was concerned with the belltower of the monastery referred to. Basic data on seismic hazard and on the dynamic characteristics of the structure are presented. The engineering analysis was performed for the elastic stage, as well as for the stage corresponding to the observed, severe, damage. The way in which damage occurred is explained. The strengthening solution is referred to and some results on its ability to resist future strong earthquakes are discussed.

Keywords: historical monument, earthquake protection, restoration, earthquake resistance, dynamic analysis, damage pattern.

1. Introduction

A French - Romanian project was initiated in 1991 in relation to the rehabilitation of the Apostolache Monastery (a historical monument, built in mid-seventeenth century). The project was intended to represent a pilot study, in the frame of which know-how of both parts was to be mobilized. Besides the French participation, coordinated for the architectural part by B. Mouton and for the structural part by V. Davidovici, Romanian specialists from several institutions, coordinated by the Direction of Historical Monuments (DMASI) were active in the project. The cooperative effort started with the belltower (about 14 m. tall) of the monastery.

2. Summary of some reference data

The reference data used were related to the seismic conditions at the monastery site and to the outcome of monitoring of ambient vibration. The site is located in a zone that, according to the provisions of the earthquake resistant design code of Romania, are the most severe of the country. According to hazard estimates performed by the authors, the basic design factor is in the range of 0.16 g for 20 years, and of 0.55 g for 200 years return period. The full-scale monitoring of ambient vibration put to evidence fundamental natural periods of 0.27 s. across the wall and 0.19 along the wall of the monastery.

3. Objectives of the engineering analyses

Analyses were related to the pre-intervention and to the post-intervention stages. The objectives for the pre-intervention stage were to explain the kind of damage observed, to explain the implications of damage upon structural performance, to estimate lower bounds of material strength, to conclude on the intervention need. The objectives for the post-intervention stage were to predict the features of performance during future earthquakes and to predict the eventual damage pattern.

4. Linear dynamic analysis

The linear dynamic analysis of the structure was aimed to help in identifying some belltower characteristics and to help also in detecting the places and nature of damage to first occur in case of a strong earthquake acting upon the undamaged structure. The adoption of a stick-type modelling was considered suitable. A parametric analysis showed what natural periods would correspond to various alternative elasticity moduli of masonry and soil. It turned out that the elasticity moduli should be very low.

5. Resistance analysis

The resistance analysis made it possible to estimate the ultimate ground accelerations , corresponding to foundation uplifting, under alternative hypotheses: for the two main directions of motion and for two limiting assumptions on the structure: integrity preserved and upper part disintegrated (as observed) respectively. The accelerations ranged from about 0.3 g to 0.6 g.

6. Expected implications of the rehabilitation and strengthening solution

The solution adopted included: reconstructing of the disintegrated masonry, introduction of horizontal and vertical (weakly prestressed) mild steel tendons and intervention at foundation level, to improve ground-structure contact. According to verifications performed, the horizontal tendons adopted are sufficient to prevent future disintegration of masonry as in fig. 2. The resistance of masonry to compressive foresee is sufficient to provide capacity of resisting up to foundation uplifting. The resistance to shear forces in horizontal sections appears to be sufficient too. Under these conditions, in case of very strong ground motions, the non-linear performance should correspond essentially to transient foundation uplifting, without significant structural damage and, also, without any practical risk of overturning.

7. Additional methodological considerations

The alternative stick models used may be considered to be quite rough. The authors believe nevertheless that this way was rather well suited in this case, given the general circumstances encountered. Something like a finite element model could not have been justified under conditions of this non-homogeneity, of lack of data about the constitutive laws and of lack of programs appropriate for non-linear analyses under these circumstances. On the other hand, the fact that ground deformability was accounted for in some way, be it simplistic, was important. It would be most suitable to repeat after the restoration work the monitoring of ambient vibration, which could provide most valuable information on the effectiveness of work undertaken.

8. Final considerations

The experience of this case showed again how important it is to try to develop models to account for the behaviour of structures on the brink of collapse. Even in case of quite rough modelling, there is a chance to obtain more realistic results than by using sophisticated linear models.