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BUILDING CONSTRUCTION UNDER SEISMIC CONDITIONS IN THE BALKAN REGION

VOLUME 5

REPAIR AND STRENGTHENING OF REINFORCED CONCRETE, STONE AND BRICK-MASONRY BUILDINGS
VOLUME 5

REPAIR AND STRENGTHENING OF REINFORCED CONCRETE, STONE AND BRICK-MASONRY BUILDINGS

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PREFACE

The Regional Project, "Building Construction under Seismic Conditions in the Balkan Region", UNDP/UNIDO RER/79/015, has been carried out with the participation of the Governments of Bulgaria, Greece, Hungary, Rumania, Turkey and Yugoslavia, and with the United Nations Industrial Development Organization acting as Executing Agency for the United Nations Development Programme. Mr. J.G. Bouwkamp served as Chief Technical Advisor.

Within the framework of the Project, a set of seven Manuals has been produced, reflecting to a considerable extent the experience of the participating nations in earthquake resistant design and construction. These Manuals were developed by the National Delegates of the Project Working Groups, the Chief Technical Advisor and the Consultants.

The following Manuals have been prepared:

Volume 1: Design and Construction of Seismic Resistant Reinforced Concrete Frame and Shear-Wall Buildings
Volume 2: Design and Construction of Prefabricated Reinforced Concrete Building Systems
Volume 3: Design and Construction of Stone and Brick-Masonry Buildings
Volume 4: Post-Earthquake Damage Evaluation and Strength Assessment of Buildings under Seismic Conditions
Volume 5: Repair and Strengthening of Reinforced Concrete, Stone and Brick-Masonry Buildings
Volume 6: Repair and Strengthening of Historical Monuments and Buildings in Urban Nuclei
Volume 7: Seismic Design Codes of the Balkan Region.

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The financial support of the United Nations and the Governments of Bulgaria, Greece, Hungary, Rumania, Turkey and Yugoslavia should hereby be acknowledged. Also the cooperation of the National Science Foundation of the USA is noted with appreciation.

The Project has been directed by the Project Coordinating Committee. The membership of the Committee was as follows:

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O.A. Nordstrand - UNDP, Athens, Greece, Resident Representative and UNDP Principal Project Representative,
E. Csorba - UNIDO, Vienna, Austria, Senior Industrial Development Officer, and
J.G. Bouwkamp - UNIDO, Thessaloniki, Greece, Chief Technical Advisor.

DISCLAIMER

The material contained in these Volumes includes detailed findings in earthquake engineering - particularly, objective evaluations of causes and effects in earthquake damage - and in the seismic and geologic characteristics of the physical environment.

The material reflects the interpretation and opinions of the authors and contributors and does not necessarily represent the viewpoint of the United Nations Industrial Development Organization, the United Nations Development Programme, the participating Governments and the National Science Foundation of the USA. The above mentioned Governments and Organizations - while providing for the presentation of these Volumes in the public interest and for their obvious informational value - assumes no responsibility for any views expressed therein.

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NOTE

This Manual is the fifth volume of the seven volumes developed under the UNDP/UNIDO Project RER/79/015 "Building Construction Under Seismic Conditions in the Balkan Region" and was prepared by the Project Working Group E on "Repair and Strengthening of Buildings". It is aimed to provide engineers in the Balkan region, as well as in other earthquake-prone countries, with information pertinent to the repair and strengthening of earthquake-damaged buildings by drawing on the experience gained following some recent strong earthquakes in the Balkan region and elsewhere.

The Manual gives mainly qualitative instructions for repair and strengthening procedures. Only in the Case Studies have quantitative results been presented, thereby reflecting in general the existing practices in the Balkan countries. However, it is fully recognized that further improvements in the repair and strengthening process require further analytical and experimental research. The cooperation between the Balkan countries will form an essential element in advancing this technology.

The contents of this Manual are based on the National Reports of the participating countries and were developed in Working Group Meetings together with the Project Chief Technical Advisor and the Consultant. The Working Group met three times, namely, in Titograd (in April 1982 and December 1982) and in Thessaloniki (in March 1983). During the first meeting, the scope of the Manual was defined and a general outline for the preparation of the National Reports was established. During the second meeting, the National Reports were presented in a two-day joint seminar, together with Project Working Group D on "Post-Earthquake Damage Evaluation and Strength Assessment of Buildings." Subsequently, together with the Consultant, the members reviewed the reports and formulated the Manual contents. During their third meeting, a first draft of the Manual was reviewed and a final text was agreed upon.

The Working Group consisted of National Delegates of the participating countries with Dr. Predrag Gavrilovic, Professor, Institute of Earthquake Engineering and Engineering Seismology, Skopje, Yugoslavia, serving as Convener. Other members of the Working Group were, Dr. Nicola Ignatiev, Professor, Higher Institute of Architecture and Engineering, Sofia, Bulgaria; Costas Syrmakezis, Dr. Civil Engineer, Lecturer, National Technical University, Athens, Greece; Pavlos Kremezis, Civil Engineer, Athens, Greece; Dr. Peter Nedli, Assistant Professor, BME, Budapest, Hungary; Dr. Nicolas Laszlo, Technical Director, Design Institute for Buildings and Town Planning (ISLGC), Bucarest, Rumania; and Dr. Günay Ozmen, Professor, Istanbul Technical University, Istanbul, Turkey. Consultant of the Working Group was Mr. Loring Wyllie, H.J. Degenkolb Associates, Engineers, San Francisco, USA.

Professor Jack G. Bouwkamp, University of California, Berkeley, California, USA, served as Project Chief Technical Advisor and participated in all Working Group discussions.

The Working Group would like to acknowledge the contribution of the Project Working Group C on "Stone and Brick Masonry Structures" in providing material pertinent to the repair and strengthening of masonry buildings.
1. INTRODUCTION

Experience from recent earthquakes has demonstrated that structures which have been properly designed and constructed are able to withstand severe earthquakes without collapse. However, these same earthquakes have shown that old buildings as well as buildings of recent construction can be seriously damaged or can collapse causing loss of life to the occupants. Studies of the structural performance during these recent earthquakes have clearly demonstrated that structural systems must not only have sufficient strength to resist lateral forces, but they also must have sufficient ductility, or the ability to maintain their integrity when stressed beyond their yield point in order to protect human life.

Repair is the reestablishment of the initial strength of damaged structural members and the reestablishment of the function of damaged nonstructural elements. Properly repaired structural members will possess approximately the same strength as before they were damaged but will probably have a somewhat reduced stiffness in concrete and masonry structures due to very fine cracks which are caused by the earthquake and are impossible to restore. Repair of nonstructural elements, such as partitions or ceilings, consists simply of adequate replacement or patching to return the function or usage of that element.

Strengthening is the judicious modification of the strength and/or stiffness of structural members or of the structural system to improve the structure's performance in future earthquakes. Strengthening generally includes increasing the strength or ductility of individual members or introducing new structural elements to significantly increase the lateral force resistance of the structure. On occasion, strengthening can also involve making selected structural members weaker to improve the interaction of the structural members and prevent premature failure of a weaker adjacent member.

Each structure is a unique system and its damage in an earthquake will be different from other structures, requiring custom repair and strengthening solutions and details. The decision of the rehabilitation (repair and/or strengthening) method and the appropriate construction technique depends on many factors, such as local site conditions, type and age of the structure, type and degree of damage, available time, equipment and staff for specific rehabilitation work, architectural requirements, cost, and the required level of seismic safety. The decision-making is one of the most responsible tasks in the rehabilitation process of a damaged structure.

Immediately following an earthquake, an Emergency Earthquake Damage Evaluation, as discussed in Section 2.1, will be performed to determine which buildings cannot be safely occupied. Those structures judged hazardous or of questionable safety will require further investigations. A preliminary investigation, as discussed in Section 2.2, will determine suitable emergency measures for temporary protection as outlined in Section 3. Additional investigations, discussed in Section 4.1 combined with appropriate criteria covered in Section 4.2 will lead to a scheme of repair and/or strengthening. Sections 5 through 8 discuss repair and strengthening materials and techniques generally used in concrete and masonry structures.
This manual, based on experience gained within the Balkan Region, is intended to provide guidance to designers for repair and strengthening of structures for seismic resistance. While repair is generally a function following a damaging earthquake, strengthening can be done either prior to or following an earthquake.
2. POST EARTHQUAKE DAMAGE EVALUATIONS

2.1 Emergency Earthquake Damage Evaluation

Immediately after a damaging earthquake, an initial evaluation of each structure will be made by official inspection teams to determine quickly the general level of damage to the structure and if the structure is safe for continued occupancy. This evaluation is described in detail in another manual of this series.

Based on this initial evaluation each examined structure may be in one of the main categories labeled as follows:

- **Green** - This category is for buildings whose original seismic capacity has not been decreased and which do not appear to pose any danger to human life. The buildings are immediately usable and the entry unlimited. These buildings may have sustained slight damage requiring repair.

- **Yellow** - Buildings in this category have decreased seismic capacity. Limited entry at Owner's risk is permitted but not usage on a continuous basis. Buildings should be repaired and/or strengthened. The need for supporting and protection of both the building and its surroundings should be considered.

- **Red** - Buildings in this category are unsafe as subject to sudden collapse. Entry is prohibited and building surroundings should be protected. Decision for demolition will be made on the basis of a more thorough inspection after investigating technical possibilities for repair and/or strengthening and their economic justification.

The Preliminary Investigation that is described in the following pages follows this initial evaluation, is an independent and more thorough evaluation and is performed by a design engineer acting for the owner of the building. The first phase of this preliminary investigation is to determine in detail the nature and degree of damage and the need for emergency, temporary bracing or shoring. Following phases of investigation involve detailed inspection of the damage so repair and strengthening procedures can be designed and detailed.

2.2 Preliminary Investigation

The main purpose of the preliminary investigation of the state of the structure is to determine in detail the nature and degree of damage and to design and install emergency measures for temporary support to avert the risk of casualties and injuries, as well as to minimize the possible material losses in case of increased damage to the structure. The probability of repeated seismic activities during the days after the first shock is quite great. Moreover, increased damage resulting from earthquake effects is often
due to the dead and live loads where the continuity of the stress path of the forces to the foundations are partially or fully interrupted. The preliminary investigation will also be utilized in determining repair and/or strengthening measures.

The preliminary investigation and the evaluation of the structure may require removal of some non-structural components, coatings, concrete covers, etc. All the deficiencies and damage, such as cracks, yielded reinforcement, excessive deformations, connection failures, etc., should be registered and described. An assessment must be made for their effect on the strength and capacity of the structural members to resist the seismic forces and the dead and live loads. The assessment of the capacity of the structure in this preliminary investigation depends exclusively on the professional judgment and the experience of the engineer. As appropriate, simplified analytic checks should be performed. The mechanical characteristics of the structural materials may be assumed on the basis of a visual evaluation.

The condition of the non-structural components should be assessed on the basis of their eventual performance with respect to the structural system. Damage in water supply and drainage systems, which may cause saturation and settlement of the soil under the foundations, must also be considered in the evaluation.

Based on the preliminary investigation and the evaluation of the damage and the structural design of the structure, the need for emergency measures for temporary protection should be determined. The results of this preliminary investigation should indicate the type of temporary protection of the structure that is appropriate. The structure may be so hazardous that the preliminary investigation concludes that total or partial demolition is most appropriate. Time permitting, appropriate methods of repairing the structure and the need for strengthening the structure may be noted, but their evaluation will follow later as discussed in Section 4.
3. EMERGENCY MEASURES FOR TEMPORARY PROTECTION

3.1 General

Immediate temporary support is recommended for buildings which are severely damaged but did not collapse after an earthquake. Severe damage may appear in the form of a rupture of a column, or serious cracking of load bearing walls, etc. Immediate measures can relieve damaged elements of their load by means of additional temporary members and thus protect the structure against a future aftershock or the effects of gravity loads on severely damaged elements.

The purpose of the temporary protection is to provide temporary strength or support for those damaged elements and connections on which the safety of the whole structural system depends. The measures for temporary protection must also provide safety for the people in the areas adjacent to the damaged building, such as in adjacent streets, sidewalks and yards. It also provides safety to workmen making repairs and installing strengthening provisions. Supporting the structure should be implemented when repair and/or strengthening of the structure is foreseen, as well as in special cases where continued functioning of the building is necessary. Supporting should not be planned in cases of obvious danger for working people installing temporary protection and demolition should be ordered for structures of such hazard.

Providing vertical support for failed or severely damaged columns or bearing walls is the first consideration in installing temporary protection. The vertical support is obviously required in the story where the vertical member is damaged. Providing temporary vertical support is sometimes feasible in only a single story, as shown in Figure 3.1, but the shear strength of the

![Fig. 3.1](image-url)
supported beams, indicated as section t-t in Figure 3.1, must be carefully checked to insure that the temporary support will be effective. Providing temporary support of more than one or all of the stories provides a more favorable situation and increased safety, as shown in Figure 3.2. Supporting all stories drastically reduces the shear stress in the supported beams at sections t-t at both sides of the damaged vertical element. The designer should pay special attention to the transmission of the temporarily supported loads to the adjacent elements as well as to the soil beneath the structure. Temporary foundations may be required. The distance between the supports and the damaged member must be a minimum, however, it should provide enough unobstructed space for the eventual repair work or replacement of the column or load bearing wall. When basements are present, the designer should also consider the possibility of lateral loads on basement walls which can cause lateral movement resulting in settlement of building supports.

When there is doubtful stability of the structure against horizontal forces, lateral counterforts or wall braces should be provided for walls which may fall laterally. Diagonal braces of structural frames can also be installed to provide bracing of the structure as a whole.

![Diagram of building with column or load bearing wall to be replaced or repaired](image)

Fig. 3.2
The design of emergency measures for temporary protection must usually be done quickly without the normal time period available for design procedures. Available materials must be utilized and only crude, approximate analysis are usually performed to determine the order of magnitude of loads and stresses. Due to the need for prompt action, judgment and ingenuity must replace detailed analysis. Later, when permanent repair and strengthening is performed, additional or supplemental protective measures may be required while that work is executed.

Depending on the means available, the seriousness of the damage and the size of the structure, a suitable method of supporting can be selected for vertical load support of damaged or failed members. Techniques such as industrial scaffolding, tree logs, steel profiles, or grillage logging can be installed. For lateral support of walls, or the structure as a whole, lateral wall bracing or frame bracing can be installed. The description of these methods is given in the following sections and is given only as an example to help the designer in selecting, combining or conceiving the method which is most suitable for each structure.

Providing temporary support of a damaged structure is dangerous for people installing the temporary protection. Therefore, the whole procedure should be properly organized, in order to minimize the working time of people in and under the structure. Bracing elements should be assembled and organized outside of the structure and then quickly installed in the structure. Wedging and jacking forces should be applied from a distance with appropriate safety precautions for workmen.

3.2 Methods for Supporting Vertical Loads

Industrial-Type Shoring and Scaffolding

In the case of very light loads, independent supports utilizing standard industrial-type shoring can be used with bearing capacity up to 2 tons and for heights of about 3.00m (Figures 3.3 and 3.4). In the case of minor damage
and for small loads, or in cases where flexural members (beams or mainly slabs) have to be supported, industrial-type scaffolding can be used (Figure 3.5). Scaffolding can be set-up using standard members, although it must be recognized that the rated capacity of such scaffold members can be greatly reduced with aging. Wedging is effected by use of special screw-type bolts with which all industrial-type scaffolding systems are equipped (Figure 3.6).

When wall piers between openings in bearing walls are cracked or of questionable stability, industrial-type scaffolding, timbers, logs or other elements capable of supporting compressive loads can be used for temporary support, as shown in Figure 3.7. Similar supporting members can be installed when lintels or walls are cracked above the openings in the bearing wall.

Timbers, Tree Logs and Telephone Poles

Timbers, such as tree logs, sawn timbers or telephone poles can also be used for supporting vertical loads (Figures 3.8, 3.9 and 3.10). Tree logs must be straight, of constant section as far as possible, of high compression strength wood and generally free of defects (knots, hollows, etc.). They should not be
1-Damaged part of wall; 2-temporary supporting scaffolding

Fig. 3.7

Fig. 3.8
made up of united smaller parts of trees. The efficiency of the timber or log section must be checked on the basis of an approximate estimation of the load and of the maximum permissible stress of the wood along the loading direction; the latter must be estimated taking into account the type and quality of the wood and its age. When loads are sufficient to require two timbers or logs on each side of the vertical element, use at least two approximately 25 cm-diameter logs or 20 cm sawn timbers, which should be interconnected with wood planks and/or stitching dogs, put in pairs, in the form of the letter X. The interconnecting members should be sufficient to brace the multiple timbers or tree logs and hold them together. Planks should be about 2 cm thick and 10 cm wide and well nailed at all locations. Stitching dogs are bent steel members, 10 mm minimum, which are driven into the vertical members and held in place with nails if necessary. Planks, 4 cm thick minimum, or of sufficient thickness as to ensure the rigidity of the base, must be placed under the foot of the timbers or logs. These planks should not be placed in more than one layer (Figure 3.11) and may be reinforced with steel plates if loads are high. Similar planks must be placed at the top of the timbers as well to distribute loads. Wedges will be inserted, by some of the methods described in a following section, between the upper planks and the top of the timbers.
Steel Profiles

Built up steel members or steel profiles can be used for vertical support (Figure 3.12 and 3.13) in the same manner as tree logs. Steel sections require bearing plates of sections both top and bottom and must be properly wedged.

Steel profiles assembled around the perimeter of a damaged column similar to permanent jacketing with steel profiles can be utilized for temporary support. The jacketing can be similar to Figure 3.14 and quickly installed with the

![Fig. 3.12](image)

![Fig. 3.13](image)

![Fig. 3.14](image)

*iron strap (every 300 to 600 mm)*

*L profiles 100.100.10*
intent that it will eventually form the basis of permanent repair and strengthening of the distressed column. Details should be similar to permanent jacketing as discussed in Section 6.1.2 with particular attention to bearing plates at each end of each steel profile to insure proper temporary transfer of the loads to be supported.

Timber Grillages

If wooden rail sleepers or other similar timber is available, vertical support can be effected by forming a grillage. The sleepers are placed in alternating layers to the required height. On top of the grillage wide flange steel I-beams or suitable timbers are placed. Wedging, in between the top of the steel beams or timbers and the lower side of the structure, is effected by using one of the methods discussed in section.

3.3 Methods for Providing Lateral Support

Lateral Wall Bracing

One of the major hazards following a damaging earthquake is the potential instability of exterior bearing walls of masonry, stone or concrete construction. These walls, when not adequately tied into the structure, tend to fall outward during earthquakes and aftershocks. When the wall falls outward, vertical support of the floors and roof may be lost as well as creating a hazard adjacent to the building.

Lateral wall bracing can be installed to prevent the wall from falling outward, as shown in Figure 3.15. The members of the bracing can be timbers,
logs or steel profiles. The diagonal member should be placed as an appropriate angle for stability with a suitable foundation in the ground to make the brace effective. The vertical member against the wall not only supports the wall but must be attached to the wall by lugs or adequate connection to counter the vertical component of force in the diagonal brace. The base of the vertical member should be embedded in the ground or otherwise supported for lateral loads. Obviously, the connection between the diagonal brace and the vertical member against the wall must be properly detailed to resist the applied forces. Braces of this type should be installed at frequent intervals along the wall to properly brace the wall.

1-exterior wall; 2-interior wall; 3-crack; 4-steel tensioner; 5 and 6-anchorage profiles; 7-steel plates

Fig. 3.16
Occasionally, it is impossible to install such a diagonal brace as it would prevent vehicle traffic along a narrow street or passageway. If similar buildings exist on both sides of the street, vertical members embedded at their base can be installed against both buildings with a horizontal brace being installed between the pairs of vertical members. Obviously, the horizontal member should be installed high enough so trucks or whatever passes beneath will not hit the horizontal brace. Connections of members must be adequate for all possible forces.

An alternate form of providing equivalent wall bracing is installing internal tension ties within the structure with steel anchorages and bearing elements on the outside of the wall, as illustrated in Figures 3.16 and 3.17. Many alternate details can be developed to suit the particular conditions of each structure.

1-exterior wall; 2-interior wall; 3-crack; 4-steel tensioner; 5-angle 50.50.5 mm; 6-steel plates; 7-steel profiles; 8-steel plates; 9-tensioner coupler

Fig. 3.17
Frame Bracing

When frame buildings have been damaged in an earthquake, it is frequently necessary to install some form of temporary lateral bracing to prevent the building as a whole from collapsing during the continued seismic activity which always follows the initial shock. Sometimes, for small buildings, lateral wall bracing or exterior counterforts can be installed which will effectively brace the entire structure. However, for many concrete framed structures, especially those without infilled walls in the lower story or with heavily damaged infilled walls in a particular story, internal diagonal frame bracing should be installed.

Frame bracing can consist of timbers, tree logs or steel profiles of sufficient strength considering their potential for buckling. The bracing members are installed on a diagonal between columns on a frame line, as shown in Figure 3.18. Bracing members can also consist of several more slender elements laced together to form a built-up compression member. The bracing members must be properly wedged or shimmed at both ends for proper bearing and to effectively transfer forces through the connection. The shear capacity of both the column and beam should be checked at the brace location to insure that sufficient strength exists to resist the components of force from the brace and thus make the brace effective.

The member of braces to be installed and their location is based on the judgment of the engineer. The location of the braces should be such as to provide a balanced system considering any shear walls which are still effective at resisting lateral forces. If the building is leaning slightly, additional braces should obviously be installed to counter the direction of permanent deformation. It may be necessary to install bracing in both directions of the building and in more than one story, based on the damage that the structure has sustained and the engineer's assessment of the vulnerability of the damaged structure for further seismic activity.

![Fig. 3.18](image-url)
3.4 Wedging Techniques

Wedging is essential for all temporary supports in order to transfer the loads from the damaged member to the new support system. For wedging temporary supports of any kind, several methods can be used, such as ordinary wooden wedges with suitable securing devices, mechanical jacks, hydraulic jacks or hydraulic flat jacks.

Wooden wedges can be used for supporting compressive forces, if the application of the impact force when driving the wedges tight does not create problems for the safety of the structure and the technicians working in it. Wedges must be used only if it is certain that no horizontal forces are acting on the member being wedged. Otherwise, the friction forces may not be sufficient to hold the wedge and the wedging may become useless. Figure 3.19 illustrates the forces acting on a wedge. Wedges must be made of dry, hard timber, be free of knots and have fibers running at right angles to the longitudinal axis of the wedge (Figure 3.20). It is important to insure the

\[ P = H \frac{1 - \tan \theta}{\tan \theta} \]

\[ f = \tan 10^\circ \]

Fig. 3.19

Fig. 3.20
immobility of the wedges by nailing one onto the other, or by other devices preventing sliding (Figure 3.21). Friction cannot be relied upon to prevent slippage. Wedges must also be fully driven to be in full bearing.

Mechanical jacks (Figure 3.22) or hydraulic jacks (Figure 3.23) can be used for wedging as they ensure uniform loading and unloading. The jacks must have an adequate base area, proportional to the load transmitted, to avoid settlement or punching. The base area should also depend on the height of the jack, to provide stability against overturning. As a general guide only, it is suggested to use 50 cm$^2$ base area per ton of load; the free length of the screw or extension of the jack should not exceed 20 cm.

In case of hydraulic jacks, the equipment comprises also an oil pump and an operation panel if many connected pumps are to be used in parallel. By connecting hydraulic jacks in parallel, it is possible to apply equal loads on several jacks and insure that the individual elements of the temporary support are subjected to approximately equal loads. Hydraulic jacking equipment must be calibrated so the technician can correlate the oil pressure with the jacking load which is applied. Hydraulic jacks as a method of wedging insures
the safety of the people working in the structure, as the loads can be applied by remote operation from a safe distance from the structure under repair and can be closely controlled.

Bearing members may have to be added at the ends of the hydraulic or mechanical jack to provide acceptable bearing stresses against the existing structural members.

Flat jacks, possibly remotely operated, can also be employed (Figure 3.24) for wedging. The fluid injected into the flat jacks is usually water or oil. For a permanent preservation of the jack deformation, cement grout may also be injected into the jack; pressure is then maintained while the grout gradually hardens. Flat jacks usually have a circular shape but rectangular and oblong flat jacks also exist (Figure 3.25). Figure 3.26 shows the form of a circular flat jack before and after injection. Plates of lead, cement grout or hard wood are sometimes placed between the structure to be supported and the flat jack (Figure 3.27) for full bearing. After jacking is complete, security wedging to support the loads in case of loss of pressure should be installed (Figure 3.28).

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![Fig. 3.24](image)

![Fig. 3.25](image)

rectangular flat-jack

circular flat-jack

oblong flat-jack

oil pressure

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Fig. 3.26

Fig. 3.27

lead cement grout or hard wood

Fig. 3.28

Section A-A

Flat-jacks

Security wedging
4. THE REPAIR AND STRENGTHENING DESIGN PROCESS

4.1 Criteria of Repair and Strengthening

A post earthquake evaluation of the seismic parameters of the region and the individual sites is required for the earthquake affected region and structures. The definition of the seismic parameters for the region is a prerequisite for successful accomplishment of the repair and strengthening of damaged structures. The study should include the expected maximum acceleration of bedrock for different return periods, amplification factors and to propose adequate time histories and average spectra for design of repaired structures. For more important structures, group or typical structures it is necessary to determine the seismic parameters for the considered sites and to perform field, soil investigations and geophysical studies to be used as input design parameters.

The definition of seismic criteria is correlation of the seismic force design parameters with the structural characteristics in terms of strength, deformability, ductility, etc. Each country or governmental region should establish its own criteria based on specific conditions related to the seismology and probable seismic events of the region. The criteria for repair and strengthening may simply be using the current Building Code for new construction. Other criterion can also be used. For example, Greece after the 1981 earthquake required a 50% increase in the base shear coefficient for designing repair and strengthening measures. Yugoslavia developed the following two levels of seismic criteria after the Montenegro Earthquake of 1979:

Design Level 1 - includes slight and moderate earthquakes whose expected acceleration corresponds to a return period of 50 to 100 years. For this seismic criteria level, the structure is assumed to perform in linear range up to the yield point, with required displacement ductility $D \leq 1$. The structure should not suffer structural damage with slight damage to infill walls for this level of seismic activity. Maximum relative story displacements are limited to $h/600$ for this level ($h$ is the story height).

Design Level 2 - is a level of maximum earthquake accelerations for return periods of 200 - 300 years, and for the defined acceleration level of the region or the site, the structure could suffer nonlinear deformations, with possible damage to infill walls, but without disturbance of structural stability. The displacement ductility criterion the maximum expected acceleration level is $D \leq 3$ to 4 for reinforced concrete structures. Maximum relative story displacements are limited to $h/150$ for this level.

The designer must use criteria as established above or the Codes and Regulations of the area as the minimum standard for repair and strengthening projects. For selected projects such as special structures, the designer may have to use criteria in excess of the established criteria based on the particular circumstances regarding the project. The designer may also have to establish criteria or methods to assign strength values to traditional materials which are not covered by modern Building Codes or Regulations.
4.2 Additional Investigations

During the preliminary investigation described in Section 2.2, the nature and general degree of damage was determined. Emergency measures for temporary protection were installed as appropriate based on that investigation. In order to design repair and/or strengthening measures, it is necessary to perform additional investigations and gather additional data while fully utilizing the preliminary investigation data.

Documents regarding the original construction should be compiled to the extent that they are available. This includes the designs, drawings, specifications, construction details, data on original construction material strengths, foundation and soil condition data, previous repairs or alterations, codes under which the original design was prepared, etc. The information gathered should be compared with the actual structure to confirm that the structure was built in conformance with that information. Deviations should be noted and recorded. If information is not available, then field measurements and observations must be taken to establish the conditions of the original construction. Finishes or cover may have to be removed to determine the composition of the structural members. If the foundations have suffered distress or settlement or if new foundations must be added, new soils borings may have to be obtained.

Completion of the detailed site inspection, begun in the preliminary investigation, must be completed. This operation is an essential and important phase in the process of designing repair and strengthening measures. Damage due to seismic forces most often appear in structural elements as columns, shear and infill walls, beams, beam-column joints, staircase towers, floor slabs and the connections between floors and walls and foundations. Each structural member must be inspected and the damage or lack of damage must be noted. Damaged members should be sketched and photographed and crack widths, spalled areas, bent or broken reinforcement, etc., should be recorded. Notes should be taken in clear fashion so that repairs can be properly designed in the engineer's office without having to continually revisit the damaged building. Photographs of the damaged members is a convenient method of recording the damage as well as providing a permanent record of the damage. The damage of each individual member can be recorded on a single sheet of paper with photographs affixed when they are developed. It is usually necessary to remove building finishes to determine the extent of the damage. Damage and location of non-structural elements should also be recorded, as damage will require repair procedures. The location of non-structural elements may affect strengthening solutions developed for the structure. All of this damage must be recorded, including the degree of damage, as repairs will be based on the data. Where appropriate, an evaluation of the remaining strength of damaged members can also be estimated and noted.

The present status, characteristics and strength of original construction materials must also be estimated during the additional investigation. This can be accomplished by various methods, by taking samples for laboratory testing and/or using suitable in-situ strength evaluation methods.

4.3 Damage Evaluation and Selection of a Repair and Strengthening Solution

Utilizing the investigation data which has been documented and the criteria for repair and strengthening, the designer must typically evaluate the damage, perform analysis to determine why the damage occurred, and develop alternate schemes to repair and/or strengthen the structure. These alternative schemes must be evaluated and the most appropriate solution selected.
The engineer must first analyze the damaged structure and thoroughly understand why the damage occurred. The force resistant paths in the building must be determined and it must be explained why certain members sustained damage while other members were essentially undamaged. It must be determined if the structure suffered due to discontinuities in strength or stiffness, due to torsional moments within the structure, due to hammering with adjacent structures or due to improper connections or details. The effects of non-structural elements such as infilled walls and appendages on the structural performance must be considered. It must be determined if members failed due to shear, compression, tension, flexure, bar anchorage, etc. This analysis is essential before any repairs can be assigned.

Calculations and analysis must be performed in order to evaluate the existing strength and stiffness of the damaged structure. The decision of the need to strengthen the structure will generally follow from these calculations. If the repaired structure without strengthening conforms to the design criteria, then strengthening will generally not be required. If the repaired strength is less than the requirements of the criteria, the strengthening will generally be appropriate in addition to the repairs of the damage.

Based on results obtained by this analysis, alternative solutions for repair and/or strengthening can be determined for evaluation of their feasibility. Several schemes should be developed schematically for strengthening the structure, as many solutions are possible, especially when new structural elements are added. Imagination and ingenuity should be exercised by the designer utilizing professional experience, as the best and most economical solution is seldom the first one conceived. Calculations for each alternative scheme must be performed to evaluate the effects of the strengthening measures. These calculations should be in sufficient detail to establish the extent of each alternative strengthening solution to insure that the criteria will be satisfied and provide a reasonable basis for comparison of the proposed schemes.

After a structural solution for repair and/or strengthening has been selected, a complete analysis with the appropriate mathematical model, according to the defined criteria, must be performed for the design. It must be recognized that despite the specific design criteria used, a future earthquake strong enough to cause damage in structures will result in some elements of the repaired structure exceeding their yield strength and be required to sustain inelastic response. Sufficient studies must be performed so it can be determined which members will fail or exceed their yield strength first. The studies must consider the consequence of these members likely to sustain inelastic stresses, for example, such response in columns may endanger the vertical support of gravity loads. Examples of structural conditions which have consistently been observed as "weak links" in concrete buildings in past earthquakes requiring particular attention include:

- short, stiff columns in comparison to other columns at the same level, possibly due to the presence of partial height infilled walls
- columns beneath discontinuous shear walls
- abrupt changes in stiffness due to changes in layout or structural system
- concrete frame joints at infilled walls
- stories which are more flexible than adjacent stories, possibly due to less infilled walls
- excessive building torsion due to inappropriate distribution of stiffness
- joint regions in concrete frames
- flat slab to column connections
- precast elements with weak connections

With a thorough understanding of the potential "weak links" in the structure, the designer can design repair and strengthening measures which will improve the response of the structure in future earthquakes. The repair and strengthening measures should establish an improved structure for seismic performance by avoiding irregularities in plan, abrupt changes in stiffness between floors and elements subject to shear or brittle failure.

The effects of strengthening elements added to the structure must be carefully evaluated to insure that they will not cause increased damage in a future earthquake. If shear walls are added, new foundations will be required not only to support the weight of the wall but particularly the overturning forces which will result. Collectors or ties will also be required to positively connect each framing level with the new shear wall. If strengthening is proposed in only a single story, the consequences of the increased strength and stiffness in that single story must be carefully analyzed and evaluated. For example, Figures 4.1 and 4.2 show two frame buildings, both damaged in...
their first story in a relatively small earthquake and subsequently strengthened only in the first floor by adding infilled concrete wall panels or wing walls to existing columns, which sustained considerable damage in their upper floors in a second damaging earthquake.

The alternative solutions for repair and/or strengthening which have been developed schematically then need to be compared and their advantages and disadvantages weighed. This comparison should result in one solution being selected for implementation. This feasibility evaluation of alternate solutions must include the following aspects:

- compatibility of the solution with the functional requirements of the structure
- feasibility of construction, including availability of materials, construction equipment and personnel with specialized training and the ability to implement the solution
- economical considerations
- sociological considerations
- aesthetics

4.4 Final Design Procedures

Final design procedures include a completion of the detailed calculations of the strengthening solution and the preparation of drawings, specifications and instructions so the work can be accomplished.
Strengthening solutions must be thoroughly designed, with adequate attachment of the new work to the existing members and structure. Appropriate calculations must supplement every step of the design to assure the designer that adequate strength, stiffness and ductility is provided.

The completed design for repair and strengthening should be clearly presented in complete construction drawings, instructions and details. The designer should keep in mind that repair techniques may be unfamiliar to some construction workers and detailed procedures and instructions must be provided. The documents must show all work that is to be performed without leaving interpretation to the Contractor and the workmen. If existing conditions are unknown, the drawings can require exposure of the unknown situation and notification of the designer for determination of the repair details. Schedules listing all columns, beams and other elements in the structure with reference to particular repair or strengthening details or procedures can be helpful in clearly defining the full scope of work.

Finally, the careful design considerations will be no better than the accuracy of the design and the execution of the repair and strengthening construction work. The design must be verified and the construction work inspected to insure that design details are properly executed. Unexpected damage or contingencies will arise requiring modification to details which the designer must review. Section 9 discusses this need for independent review and inspection. Thorough inspection is an essential part of seismic rehabilitation projects.
5. MATERIALS AND CONSTRUCTION TECHNIQUES

This section covers materials and construction techniques which are frequently encountered in the repair and strengthening of structures following a damaging earthquake. Designers may not be familiar with some of these materials or techniques and this section is intended only as a brief statement of guidance. Before utilizing any of these materials or techniques, the designer should study technical literature, obtain advice as necessary, and be thoroughly familiar with the process before including it in a design.

5.1 Conventional Cast-in-Situ Concrete

Conventional cast-in-situ concrete is often used in repair and strengthening works. In many cases, the results are unsatisfactory, mainly because of the volume change or shrinkage of the conventional cement-based concrete. The volume changes cause loss of good contact between the new concrete and the old element, thus preventing sound transfer of stress at the contact surface. In order to improve the bond characteristics and minimize the shrinkage, it is recommended to use higher strength concretes ($f_{\text{add}} = f_{\text{exist}} + 5\text{MPa}$) with low slumps and minimum water. Superplasticizers might also be used to reduce shrinkage. In cases that superplasticizers are used, a slump of about 20 cm is expected, while without superplasticizers the slump should not exceed 10 cm, using a standard Abrams cone.

Placement techniques are very important with cast-in-situ concrete to insure that the new concrete will perform adequately with the older materials. Existing surfaces which will be in contact with new cast-in-situ concrete must be thoroughly roughened and cleaned for good bonding characteristics. After anchorages are installed, forms are constructed to meet the desired surfaces. Special chutes or access holes are frequently required in the forms to allow placement of the concrete. Immediately before placement, a final cleaning of the forms is essential to remove all sawdust, etc., and the existing concrete or masonry surfaces should be moistened. The concrete should be thoroughly vibrated to insure that it completely fills the forms and that voids or rock pockets are avoided. Proper curing of the newly cast concrete is also important to prevent rapid drying of the surface.

5.2 Concrete Using Shrinkage Compensating or Expansive Cement

Concrete of this type is produced with expansive instead of conventional cements and they obtain an appreciable initial volume increase which tends to compensate the eventual shrinkage of the mix. Expansive admixtures (for example, fine iron or aluminum powder) may also be used with cement, water, sand and aggregates to produce this type of concrete. However, it is mandatory when using concrete with expansive or shrinkage compensating properties that all properties are precisely known or it is necessary to perform extensive laboratory tests to determine the properties of the concrete mix. The designer should use shrinkage compensating or expansive cement in concrete mixes only after a thorough investigation of the properties for compatibility with the existing materials.
5.3 Polymer Modified Concrete

Polymer modified concrete is produced by replacing part of the conventional cement with certain polymers which are used as cementitious modifiers. The polymers, which are normally supplied as dispersions in water, act in several ways. By functioning as water reducing plasticizers they can produce a concrete with better workability, lower water-cement ratio and lower shrinkage than conventional concrete. They can improve bond between the old and new elements. They act as integral curing aids, reducing but not eliminating the need for effective curing. By introducing plastic links into the binding system of the concrete, they improve the strength of the hardened concrete. They can also increase the resistance of the concrete to some chemical attacks. However, it must be cautioned that such polymer modified concretes are bound to lose all additional properties in case they come under fire. Their alkalinity and, thus, the resistance against carbonating will be much inferior to normal concrete. The design should use polymer modified concrete only after a thorough investigation of the properties for compatibility with the existing building materials.

5.4 Resin Concretes

In resin based concrete mixes, the cement is replaced by a two-component system, one component being based on liquid resin (epoxy, polyester, polyurethane, acrylic, etc.), which will react by cross linking with the second component, called a hardener. Resin concretes can be useful in patching small spalled areas of concrete and are not in general use for large volumes of new concrete. Resin concretes require not only a special aggregate mix to produce the desired properties but also special working conditions, since all two-component systems are sensitive to humidity and temperature.

The properties of resin concrete are as various as the number of resins offered by the industry for this purpose. However, there are some common tendencies of this relatively new construction material that should especially be taken into consideration, when using it for repair and/or strengthening works:

- Resins have a pot life which must be strictly adhered to in use so that the work is completed before the resin hardens.

- For the resin types used for construction purposes, normal reaction cannot be reached at low temperatures (below +10°C); in warm weather the heat developing during the reaction can be excessive and give rise to an excessive shrinkage of the mix.

- Although the direct bond of a resin compound on a clean and dry concrete surface is excellent, a resin concrete has generally poor direct bond on concrete, due to the fact that there can only be a point-to-point connection between the resin-covered aggregates and the old concrete. Thus, to assure a good bond it is necessary to apply a first coating of pure liquid resin onto the existing concrete surface.

- Resin concretes will commonly have a much higher strength but also a different elasticity than normal concrete; problems resulting from the different elasticity must be appropriately considered.

The designer should use resin concretes only after a thorough investigation of the properties and material limitations with the existing building materials.
5.5 Shotcrete (Gunite)

When equipment and specially trained workmanship is available, the method of shotcreting is often desirable for strengthening reinforced concrete members.

The widespread utilization of shotcrete for repair and/or strengthening projects is based on its numerous advantages. A very good bond can be obtained between the new shotcrete and the old concrete or masonry and the new reinforcement when properly applied on prepared surfaces. The strength characteristics of shotcrete are high due to the high compaction energy and to a low water/cement ratio. Shotcrete can be sprayed on vertical, inclined and overhead surfaces with a minimum of formwork.

Surface preparation before shotcreting involves a thorough cleaning and removing all loose aggregate and roughening the existing concrete surface for improved bond. Shotcrete frequently has high shrinkage characteristics and measures to prevent cracks using adequate reinforcement and proper curing is always necessary. The shotcrete surface can be left as sprayed which is somewhat rough. If a smoother surface is required, a thin layer can be sprayed on the hardened shotcrete and then reworked and finished to the required texture or plaster can be applied.

The equipment required for a minimum shotcrete operation consists of the gun, an air compressor, material hose, air and water hoses, nozzle, and sometimes a water pump. Miscellaneous small hand tools and wheelbarrows are also required. With this minimum equipment, shotcrete work can be accomplished satisfactorily. To assure a constant flow of water of satisfactory pressure, shotcrete equipment should include a water pump of piston, gear or centrifugal type. If the piston pump is used, it should feed into a receiver tank to eliminate pulsation of water at the nozzle. Modern high-proaction equipment includes considerably more investment in elaborate elevators, mixers, batchers and associated equipment. Figure 5.1 schematically illustrates the typical equipment required for shotcreting.

The normal flow of material in a shotcrete operation follows a standard pattern for all types of equipment. First, the materials must be batched, usually in quantities of approximately 50 kg of cement to 200 kg of sand. Batching equipment may be sophisticated and automatic or as rudimentary as so many wheelbarrow loads of sand per bag of cement. Quantities can be controlled by either volume or by weight, although the weight method is generally used. After batching, the material is mixed by a paddle, turbine or drum mixer and, in some cases, by a screw mixer or conveyor. The mixed material is carried in suspension by compressed air through a hose to the nozzle. At the nozzle, water is injected into the material in a number of fine streams. As the material passes through the final 20 to 30 cm of the nozzle, it is mixed with water. Mixing continues as the stream of material and water passes through the nozzle and upon impact, water mixing is complete.

5.6 Resins

Resins are conventionally used for injections and gluing of thin steel sheets. These materials are two-component systems; one component is a liquid resin (epoxy, polyester, polyurethane, acrylic, etc.) while the second component is a hardener. There is a large variety of available products with different properties, depending on the kind of components and their chemical structure, on mixing ratios, on the quantity and type of filler and sand eventually added, etc. Therefore, the desired properties have to be precisely defined, in order to select the correct formulation of the synthetic mixture.

Resins used in the repair and strengthening process must have properties
available for the intended usage. They must have adequate pot life, and good workability, but the pot life should not be too long to prevent hardening from occurring promptly. Curing needs should be compatible with the humidity and temperature of the installation. The resin material must have excellent bond and adhesion to concrete, masonry and steel, and with no reduction of adhesion after being exposed to moisture. The resin should have little or negligible shrinkage deformations and should not be used where creep deformations will be detrimental. Resins used for construction purposes will generally lose their strength above +100°C, so their usage is restricted and fire protection measures may be necessary. Resins used for injection should have a viscosity appropriate for the width of crack being injected. Resins used for gluing typically need to have high viscosity. The designer should select and use a resin only after thorough investigation of the properties of the resin for compatibility with the existing materials and its intended use.

Several different injection techniques may be used, such as low pressure injection (up to 1 MPa), high pressure injection (up to 20 MPa), or vacuum injection. Each injection technique requires a special epoxy resin adapted to the system. Some injection equipment feeds the resin and the hardener through separate hoses to the mixing head where the two components are automatically mixed in the correct proportions and injected under pressure directly into the crack. It is essential that the components be properly mixed in correct proportions.

When the width of cracks is rather small (0.1 to 0.5 mm) a resin is used without a filler. In case of wider cracks, it is advisable to add a filler,
in order to reduce shrinkage, creep and thermal phenomena. Glass or quartz powder can be used as long as the crack width is not larger than 1.0 to 1.5 mm, whereas sand should be used beyond this limit and up to 4.0 to 5.0 mm. The maximum dimension of the grains must be not larger than 50% of the minimum width of a crack and not more than 1.0 mm in any case. A resin/filler ratio of about 1:1 is normally used.

The crack must be sealed at the surface with an appropriate material to contain the liquid resin when injected. Nipples or ports are placed through the sealer at spacing dependent on the width and depth of crack to be injected, and the resin is injected from the lower nipple or port until it is observed in the next port, at which time the injection proceeds to that port, etc.

Finally, the fact that all elements repaired by resin injection or impregnation have to be protected against temperature variations and especially against fire has to be emphasized.

5.7 Grouts

Grouts are frequently used in repair and strengthening work to fill voids or to close the space between adjacent portions of masonry or concrete. Many types of grout are available and the designer must choose the proper grout for the intended usage.

Conventional grout consists of cement, sand and water and is proportioned to provide a very fluid mix which can be poured into the space to be filled. Forms or closures are necessary to contain the liquid grout until it has set. Conventional grout of this type has excessive shrinkage characteristics due to the high volume of water in the mix. Placing grout in a space of 2 to 5 cm wide will result in enough shrinkage to form a very visible crack at one side of the grouted space. Thus, conventional grouts should be used only when such cracking due to shrinkage will be acceptable.

Cement milk is formed by mixing cement with water into a fluid to place in the very small cracks. Superplasticizers are required with such mixes to maintain the water at an appropriate quantity required to hydrate the cement.

Non-shrink grouts are available for use when it is desirable to fill a void without the normal shrinkage cracks. The dry ingredients for non-shrink grout comes premixed in sacks from the manufacturer and are mixed with water in accordance with the manufacturer's instructions. There are many types of nonshrink grouts available, but designers should be aware that the cost of these materials is considerably more than that of conventional grout. The properties of grouts mixed with these materials should be known before specifying their use on a repair or strengthening project.

Epoxy or resin grouts are also available for conditions when high shear force or positive bonding is necessary across a void. Epoxy grouts come prepackaged from the manufacturer and must be mixed and used in strict accord to the instructions. Placement must be completed within the pot life of the resin before the ingredients have set. Epoxy grout generally does not shrink and provides a bonding similar to that of epoxy products.

Many other types of grouts can be created using polymer products and other newer concrete products. Shrinkage of conventional grout can be reduced using superplasticizers. The designer should become thoroughly familiar with the properties of the materials which are to be used on his project, and trial batches should be mixed and tested where appropriate.
Injections of grouts require special equipment and specially trained personnel similar to epoxy resin injections. This method is used for the repair of members that are compressed by filling the joints, cracks or gaps. It is also used in the restoration of the bearing surface of footings.

Grout injection is similar to resin injections although a cement or similar grout is used for injecting. Grout injection is generally used to fill large cracks (too large to practically fill with a resin) or fill voids. Figure 5.2 shows a schematic arrangement of equipment typically used in grout injection.

In many instances, it is inappropriate to fill a void with a fluid grout and a dry material that is packed or tamped into the void is used. Such a material is called drypack and consists of cement and sand with only a slight bit of water to moisten the dry ingredients. Drypack is placed in the void and hand-tamped with a rod until the void is filled. Drypack should be used only in sizable voids which are wide enough to allow thorough compaction by tamping. Due to its low water content, drypack generally has low shrinkage properties.

1 - mixer; 2 - agitator; 3 - grout pump; 4 - recording unit;
5 - pressure transducer; 6 - manifold; 7 - measuring unit;
8 - packer

Fig. 5.2 Grout injections
6. REINFORCED CONCRETE STRUCTURES

6.1 Repair and Strengthening of Original Structural Elements

6.1.1 Introduction

Repair of reinforced concrete elements is often required after a damaging earthquake to replace lost strength. Strengthening of reinforced concrete structural elements is one method to increase the earthquake resistance of damaged or undamaged buildings. Thus, the strength of the structures can be moderately or significantly increased and the ductility can be improved.

Depending on the desired earthquake resistance, the level of the damage, the type of the elements and their connections, members can be repaired and/or strengthened by injections, removal and replacement of damaged parts or jacketing.

Establishing sound bond between the old and the new concrete is of great importance. It can be provided by chipping away the concrete cover of the original member and roughening its surface, by preparing the surfaces with glues (for instance, with epoxy prior to concreting), by additional welding of bent reinforcement bars or by formation of reinforced concrete or steel dowels.

Perfect confinement by close, adequate and appropriately shaped stirrups and ties contributes to the improvement of the ductility of the strengthened members. Detailed consideration of the possibility of significant redistribution of the internal forces in the structures due to member stiffness changes is very important.

Jacketing with steel profiles (angles and straps) is used at the strengthening of separate members, mainly columns. The joint beam-to-column is difficult to strengthen by this technique.

Jacketing by steel encasement is implemented by gluing of steel plates on the external surfaces of the original members. The steel plates acting as reinforcement are glued to the concrete by epoxy resin. This technique does not require any demolition, it is considerably easy for implementation and there is a negligible increase in the cross section size of the strengthened members.

Jacketing by steel profiles or encasement requires special measures to be taken for fire and corrosion protection of the new incorporated steel profiles or plates. Reinforced concrete jacketing does not need such a protection but the construction procedure is more difficult.

6.1.2 Columns

The aim of rehabilitation is to improve the earthquake resistance of the buildings. Increasing column flexural and shear strength, improving column ductility and rearrangement of the column stiffness can be achieved by repair and/or strengthening techniques as discussed in this section.
Column flexural strength increases with the enlargement of the concrete area and by adding new longitudinal reinforcement. Shear strength, and especially ductility, is improved by better confinement with close transverse reinforcement - ties or steel strips. Equalizing of column stiffness by rearrangement, like separating columns from spandrel walls, improves the compatible behavior of all columns of the structure.

Damage of reinforced concrete columns without a structural collapse will vary, such as a slight crack (horizontal or diagonal) without crushing in concrete or damage in reinforcement, superficial damage in the concrete without damage in reinforcement, crushing of the concrete, buckling of reinforcement, or rupture of ties. Based on the degree of damage, techniques such as injections, removal and replacement, or jacketing can be provided.

Local Repairs

Resin injections are applied only for repair of damaged columns with slight cracks without damaged concrete or reinforcement. Epoxy resin injection batch or mixed in the head of the equipment is suitable for cracks with width from 0.1 to 5 mm as discussed in Section 5.6. The epoxy resin is injected into ports placed in drilled holes, spaced from 20 to 100 cm (Figure 6.1). Cement grout injections can be applied for larger cracks (widths from 2 to 5 mm). The injecting procedure initiates from the column bottom and proceeds upward. Strength and compactness checks should be verified by appropriate samples.

Removal and replacement should be performed for heavily damaged columns with crushed concrete, buckled longitudinal bars or ruptured ties.

When the concrete is only slightly damaged, the loose concrete is removed, the surfaces are roughened and dust is cleaned. Depending on the amount of concrete removed, some additional ties or reinforcement may be added. Before concreting, the existing column should be saturated with water as necessary.
Temporarily, the form and the concrete should be placed higher than the final repaired top level in order to compact the concrete sufficiently. After one day, the form can be removed and the fresh concrete that is beyond the normal configuration can be chipped away (Figure 6.2).

When the longitudinal reinforcement is buckled, the ties are ruptured and the concrete is crushed, total removal and replacement of the damaged parts must be carried out (Figure 6.3). If only repair is required, the original cross sections size will be maintained. If strengthening is necessary, the area of the column must be increased. Damaged and loose concrete should be removed, new longitudinal reinforcement inserted and welded to the existing reinforcement and new additional close ties placed. Non-shrinkage concrete or concrete with low shrinkage properties should be placed. Special attention must be paid to achieving good bond between the new and the existing concrete.

Reinforced Concrete Jacketing

Jacketing should be applied in cases of heavy damaged columns or in cases of insufficient column strength. Because of the increased capacity of the columns, this is really a strengthening procedure although it can also provide repair. Jacketing can be performed by means of adding reinforced concrete, a steel profile skeleton or steel encasement.

Reinforced concrete jacketing according to the available space conditions around the column can be performed by adding jacketing to one, two, three or four sides of concrete column sections (Figure 6.4). It is strongly recommended that columns be jacketed on all four sides for best performance in future earthquakes. In order to achieve the best bond between the new and the existing concrete, four-sided jacketing is also most desirable. In case one,
two or three sided jacketing is all that is possible, the concrete cover in the jacketed parts of the existing column must be chipped away so new ties can be welded to existing ties. In case of a four-sided jacketing, only roughening of the surface of the existing column may be required unless greater load transfer is desired.

1 - existing non-damaged concrete; 2 - existing damaged concrete; 2 - new concrete; 4 - buckled reinforcement; 5 - new reinforcement; 6 - new ties; 7 - welding; 8 - existing ties; 9 - existing reinforcement

Fig. 6.3
Fig. 6.4

Jacketing only in the story space without reinforcement penetrating through the floor structure can improve the local axial and shear strength of the column (Figures 6.4 and 6.5.a). However, the flexural column strength is not improved and the column-to-beam joint is not strengthened. Thus, the total frame structure may show poor behavior in future earthquakes. Jacketing only within the story is a local strengthening which does not improve seismic response unless significant shear walls are also added. Adequate column flexural strength can be achieved by passing the new longitudinal reinforcement through holes drilled in the slab and placing new concrete in the beam-column joint region (Figure 6.5.b). Special attention should be paid to the good confinement of the longitudinal reinforcement in the region of the floor beams.
1 - slab; 2 - beam; 3 - existing column; 4 - jacket; 5 - added longitudinal reinforcement; 6 - added ties
In the case of a one-sided jacket, adequate connection between existing and new concrete must be achieved by good detailing and closely spaced, well anchored, additional transversal reinforcement (Figure 6.6). The following solutions can be applied:

- Anchorage by ties to the existing longitudinal reinforcement (Figure 6.6.a). Welding is not necessary, but chipping free space for passing the hooks of the additional ties is necessary.

- Welding of additional ties to the existing column (Figure 6.6.b). The concrete cover in tie region must be removed and every new tie must be welded to the existing one.

- Connection by bent bars welded to the longitudinal reinforcement (Figure 6.6.c). The concrete must be chipped only in the welding region up to the longitudinal reinforcement. By this way, concrete keys capable of transmitting shear forces are formed. The bent bars allow direct transfer of forces between longitudinal reinforcement. In lieu of welded bent bars, a vertical steel plate can be welded between existing and new longitudinal bars.

Similar detailing is applied in case of two or three-sided lateral jacketing.

In the usual case of four-sided jacketing, several solutions are possible as presented in Figure 6.7, as follows:

- Jacketing can be achieved with welded wire fabric and new concrete cover. This solution improves the local column ductility, but the flexural column strength is not significantly increased because of the impossibility of the wire fabric passing through the floor structure. The fabric should completely encase the column with a long overlap or consist of two sections with an adequate overlap on two opposite sides.

- Jacketing with connecting bent bars (Figure 6.7.a). The added longitudinal reinforcement is connected to the existing reinforcement by welding of bent bars. This jacket type is applied for large column cross sections where the middle reinforcement cannot be confined by new ties.

- Jacketing with ties (Figure 6.7.b). Concentration of the newly-added longitudinal reinforcement at the corners of the cross section allows an adequate confinement of all longitudinal bars. The jacket should be of sufficient thickness with closely spaced ties to provide confinement. With the new longitudinal reinforcement passing through holes drilled in the slab, this procedure provides a continuous connection of the jackets to the upper story and lower story columns.

Good confinement can also be achieved by jacketing with circular hoops (closed ties) and a new circular concrete section. In case of wide beams, the longitudinal reinforcement cannot pass through the floor structure. When the floor structure is a flat plate (beamless) or has relatively narrow beams, connection of the added longitudinal reinforcement through drilled holes in the slab is possible (Figure 6.8). Special attention must be paid to the adequate anchorage and splicing of the column ties or hoops.

Reinforced concrete jacketing should also conform to the following provisions:

- The strength of the new materials must be equal or greater than those of the existing column. Concrete strength should be at least 5 MPa greater than the strength of the existing concrete.
1 - existing column; 2 - jacket; 3 - existing reinforcement;
4 - added longitudinal reinforcement; 5 - added ties;
6 - welding; 7 - bent bars

Fig. 6.6
1 - existing column
2 - jacket
3 - key
4 - bent bars
5 - added reinforcement
6 - ties
7 - welding
8 - alternative corners

Fig. 6.7
1 - existing column; 2 - jacket; 3 - added reinforcement; 4 - hoop; 5 - drilled holes

Fig. 6.8
- The thickness of the jacket should be at least 4 cm for shotcrete application or 10 cm for cast-in-situ concrete.

- The area of longitudinal reinforcement should not be less than 0.01 and no more than 0.06 times the gross area of the jacket section. The reinforcement should not be less than four bars for four-sided jacketing and bar diameter should be at least 14 mm.

- Ties should be arranged so that every corner and alternate longitudinal bar should have lateral support provided by corners of the ties with an included angle of no more than 135 degrees. No intermediate bar should be farther than 10 cm from corners of the ties. In some cases, it will be necessary to drill into the core of the existing column and epoxy hooked ties into the hole or drill completely through the existing column core to install new ties.

- The diameter of ties (except welded wire fabric) should be minimum 8 mm, but not less than 1/3 of the longitudinal bar diameter.

- Vertical spacing of ties shall not exceed 20 cm, while the spacing close to the joints within a length of 1/4 of the clear height should not exceed 10 cm. In addition, it is advisable that the spacing of ties should not exceed the thickness of the jacket.

Jackets can be installed either as conventional or special cast-in-situ concrete or by shotcrete (gunite). For both methods, the existing concrete surface must be thoroughly roughened by chipping or heavy sandblasting and cleaned of all loose material, dust and grease. The surface should be thoroughly moistened before placing the concrete or shotcrete. Section 5.5 discusses shotcrete procedures and application.

**Steel Profile Jacketing**

Steel profile skeleton jacketing consists of four longitudinal angle profiles placed one at each corner of the existing reinforced concrete column and connected together in a skeleton with transverse steel straps (Figure 6.9). They are welded to the angle profiles and can be either round bars (minimum diameter of 12 mm) or steel straps (minimum size of 25/4 mm). The angle profile size should be no less than L 50/50/5. Gaps and voids between the angle profiles and the surface of the existing column must be filled with non-shrinking cement grout or resin grout. A covering with concrete or shotcrete reinforced with welded fabrics is efficient for corrosion or fire protection. Tight bearing between the angle profiles and the floor structures, important for transmitting the forces, is achieved by an angle profile collar formed around the column perimeter directly in contact with the surfaces of the floor structures. In general, an improvement of the ductile behavior and an increase of the axial load capacity of the strengthened column is achieved. However, the stiffness remains relatively unchanged. This procedure can also be used for temporary support as discussed in Section 3.2.

**Steel Encasement**

Steel encasement is the complete covering of the existing column with thin steel plates (Figure 6.10). Steel encasement offers the possibility of only a small increase in column size. Steel plates (with thickness from 4 to 6 mm) are continuously welded together and located at a distance from the existing column. The voids between the encasement and the column should be filled with non-shrinking or expanding cement grout or concrete. The circular encasement is most effective (Figure 6.10.a) as the circular plate provides
confinement through hoop stresses. Rectangular encasement can be realized by welding two L-shaped steel sheets (Figure 6.10.b.) or by welding of four vertical steel strips to four angle profiles (Figure 6.10.c). Ductility and axial force strength can be considerably increased locally by steel encasement (especially by circular one). However, the flexural strength of a frame structure cannot be improved by this method because it is impossible to pass the steel encasement through the floor structure. Special measures must be provided for fire and corrosion protection.

1 - existing column; 2 - steel angle profile; 3 - steel plate; 4 - supporting plate; 5 - angle profile

Fig. 6.9
1 - existing column; 2 - new concrete or grout; 3 - steel incasement; 4 - steel angle profiles; 5 - steel plate; 6 - welding

Fig. 6.10
Design Considerations

Design calculations for repair and/or strengthening of columns are necessary in verifying analytically that the strengthening provides sufficient axial, shear and flexural strength as well as improving the ductility. Calculations are also need to evaluate the stiffness of the strengthened column. The extent of the design calculations depend on the degree of damage and the strengthening level which must be achieved. In considering flexural strength, both the added reinforcement area and concrete area must be considered. Sufficient shear strength should be provided by proportioning the additional ties or welded wire fabric, which also increases the ductility of the strengthened column. The enlarged sections of repaired or strengthened columns can result in considerable stiffness change of the different members, causing a redistribution of seismic moments and affecting the seismic forces in different parts of the building structure. The change in stiffness can also affect the seismic design forces. Therefore, the most realistic evaluation of the stiffness of the strengthened columns is very important.

The interaction between both the existing and the newly added parts of the column are strongly influenced by the bond properties of both the existing and the newly-added materials. If the bond is perfect and no slippage occurs between the existing and newly-added parts, the strengthened column acts like a monolithic block and the greatest strength and stiffness are developed. Both the loading time history that is the temporary axial force unloading during the strengthening procedure and the specific deformation material properties (including creep and shrinking phenomena) influence the distribution of the internal forces between the existing column and the newly-added jacket and should be considered in the calculations. For simplicity in strength calculations, an appropriate assumption that can be used if the column was not severely damaged and good bond is achieved, is to assign the dead loads to the original column section while utilizing the composite section of original and jacket section to resist live loads and seismic loads.

The stiffness of a strengthened column should be evaluated by expressions taking account of the specific behavior of the compound members. If good bond between the existing column and the new concrete of the jacket exists, the stiffness of the strengthened column should be determined as those of monolithic members with equivalent composite cross section. If the bond is questionable or the strength of the concrete of the existing column is partially lost, the contribution of the existing column must be reduced by appropriate assumptions. Nevertheless, the stiffness of the strengthened column should be no less than the sum of the stiffness of the existing column and that of the new jacket, taken as separate members. In the limit case, when the existing column is entirely ruptured, the member stiffness should be evaluated by taking into account the new jacket stiffness only as the repaired original column may contribute negligible stiffness.

Because of the uncertainties of the real stiffness values of strengthened columns, the designers should make appropriate stiffness assumptions in calculations to achieve a suitable final design. For most structures, a practical approach usually results using an assumption of monolithic member stiffness values. Calculations using extreme values of stiffness may be appropriate to evaluate the relative stiffness of columns and beams framing into specific frame joints. For complex structures, more detailed calculations considering the variations in stiffness assumptions can be used.
6.1.3 Beams

The aim of repair and/or strengthening of beams is to provide adequate strength and stiffness of damaged or undamaged beams which are deficient to resist gravity and seismic loads. It is very important that the rehabilitation procedure chosen provides proper strength and stiffness of the beams in relation to adjacent columns in order to avoid creating structures of the "strong girder - weak column" type which tend to force seismic hinging and distress into the column, which must also support major gravity loads.

Depending on the type of damage (cracking, crushing of concrete, rupture of reinforcement or ties) and the necessary level of strengthening, the techniques for repairing and strengthening beams are quite similar to techniques used for columns.

Local Repairs

Injections are applied for repair of damaged beams with slight cracks only. Epoxy or cement grout injections are made according to the recommendations in Section 5.6 and 5.7 and similar to such repairs for columns.

Removal and replacement should be applied when heavy damage like crushing of concrete, deterioration of bond or rupture of reinforcement occurs. Before the removal of crushed concrete or rupture reinforcement, the damaged beam must be temporarily supported. The replacement procedure of beams is similar to that of columns. More attention must be paid to compacting the new concrete under existing beams or slabs which is extremely difficult if placement access is not provided from the top surface of the beam.

Reinforced Concrete Jacketing

Reinforced concrete jacketing can be performed by adding concrete on one, three or four sides of a beam. In order to create adequate bond between original and new concrete and for welding of the added reinforcement to the existing reinforcement, the concrete cover must be chipped away. An irregular shaped concrete surface by the roughening of the concrete combined with anchoring by welded stirrups provides good shear and tensile connection of the jacket to the existing beam.

According to the type of strengthening (flexural or shear) the longitudinal or the transverse reinforcement predominates. Reliable anchorage of longitudinal reinforcement bars in joint areas by sufficient development length or reinforcement welding to steel anchor plates or profiles is of great importance. Adequate shear and ductility improvement should be provided by additional stirrups distributed on all sides of the existing beams. The legs of the stirrup should penetrate through drilled holes in the slab in the top part of the jacket, where they must be sufficiently anchored.

One-sided jackets (Figure 6.11) or adding strength only to the beam soffit should be used only when it is necessary to increase the flexural strength in the mid-span of a beam. The connection between the existing and the new longitudinal reinforcement is achieved by welded connection bars (See Figure 6.11). The concrete cover should be chipped away to the existing longitudinal reinforcement and higher at existing stirrups. Additional stirrups welded to the existing ones provide the connection between the existing beam and the newly-added concrete. The longitudinal reinforcement bars should be anchored in the support region, which may be by welding of the reinforcement to a collar of steel angle profile attached to the top of the column.
Four-sided jacketing (Figure 6.12) or encasement of a beam adds considerable flexural and shear strength because of the increase of the reinforcement area and concrete cross section dimensions (depth and width). The additional longitudinal reinforcement should be connected with the existing reinforcement by diagonally welded bent bars or small steel plates. The stirrups pass through holes drilled in the slab and surround the whole beam. These holes can also be used to place the concrete in the part of the jacket beneath the slab. Additional reinforcement for negative bending moments must be added over the slab surface in the beam region and outside of the existing column. Special attention must be paid to the anchorage of the longitudinal bars in the joint region of the column jacket.
Jacketing on three sides of the beam can also be installed beneath the soffit of the slab, as shown in Figure 6.13. Shotcrete is the most feasible method of installing this type of jacket and its primary weakness is the anchorage of the new stirrups at the top of the sides of the jacket. This detail is inferior to that shown in Figure 6.12 as the effectiveness of the detail depends on the dynamic strength of the power driven nails and the stiffness of the strand to provide effective anchorage for the new stirrups. Increased strength might be achieved by using a continuous steel plate with epoxy resin bolts installed in the concrete with the new stirrups welded to or hooked around the steel plate.
Reinforced concrete jacketing should conform to the following provisions, as well:

- The strength of the new materials should not be less than those of the existing beam.

- The thickness of the jacket should not be less than 4 cm for shotcrete application or 8 cm for cast-in-situ concrete.

- The beam reinforcement of moment resisting frames should be continuous in the top and bottom of the beam and not less than 0.005 times gross area of beam at support regions.

- Top and bottom reinforcement should be anchored within column joint area with full development length, beginning from the face of the column, or continuous through the joint region.

- At support regions for a length equal to 4 times the overall beam height, the stirrup spacing must not be more than 1/4 of the beam height. Outside this region, the stirrup spacing can be doubled.

Partial shotcrete jacket on beam

Fig. 6.13
Repairing Gravity Load Capacity of Beams

Steel rods can be used for improving the shear resistance of damaged or undamaged beams. It can be performed by vertical external clamps (Figure 6.14.a) or by diagonal ones (Figure 6.14.b). The clamps consist of round rods with threads at the end which are tightened with nuts. At the beam, bottom vertical clamps are fixed on angle profiles, but diagonal clamps are welded to longitudinal reinforcement to resist the longitudinal component of force. If load reversals are anticipated, four-sided jacketing is the preferred method of strengthening.

1 - existing beam; 2 - steel clamp; 3 - steel plate; 4 - nut; 5 - angle profile; 6 - welding

Fig. 6.14
Steel plate reinforcement is a new technique which can be used for beams subject primarily to static loading to improve their shear strength or midspan flexural strength. The steel external plates are attached to concrete surfaces of the reinforced concrete members by gluing with epoxy resin. During the epoxy hardening, the steel plates must be clamped to the concrete member. It is recommended that the steel plates also be anchored by either nails shot into the concrete or anchor bolts (wedge or epoxied). Steel plates have a thickness from 20 to 10 mm. It is advisable for the beam to be smoothed with a thin layer of expansive cement mortar for plates with thickness more than 3 mm. In this case, wedge anchor bolts must be applied. Special attention must be paid to corrosion and fire protection, especially considering the total loss of epoxy resin strength at temperatures higher than 250°C. This procedure is not recommended for beams subject to cyclic loading due to earthquake forces.

Design Considerations

Adequate flexural and shear strength must be provided and verification by design calculation is essential. Calculation procedures are similar to those discussed in Section 6.1.2 for columns. It is important that the range of stiffness values for the beam and column be investigated for moment resisting frames subject to seismic forces to insure that potential hinging will occur in beam rather than in column. The beams must not be made too stiff with respect to the adjacent columns.

6.1.4 Beam-Column Joints

The most critical region of a moment resisting frame for seismic loading, the beam to the column joint, is undoubtedly the most difficult to strengthen because of the great number of elements assembled at this place and the high stresses this region is subjected to in an earthquake. Under earthquake loading joints suffer shear and/or bond (anchorage) failures. Severe earthquake excitations can produce plastic hinges in the beams at the column faces. As a result, cracks develop throughout the overall beam depth. Bond deterioration near the face of the column causes propagation of beam reinforcement yielding into the joint and a shortening of the bar length available for force transfer by bond causing horizontal bar slippage in the joint. At the interior joint, the beam reinforcement at both the column faces is under different stress conditions (compression or tension) because of the opposite signs of the seismic bending moments. Therefore, in this case particular attention must be paid to the reliable anchorage of the beam reinforcement in the joints.

Local Repairs

Epoxy injections can be applied for repair of damaged joints with slight to moderate cracks without damaged concrete or bent or failed reinforcement. Restoration of bond between reinforcement and concrete by injections is inadequate and unreliable. Epoxy injections are to be performed according to the recommendations in Section 5.6.

Removal and replacement should be applied in cases of crushed concrete, deteriorated bond or ruptured reinforcement. Before the removal, the damaged structure must be temporarily supported. The replacement procedure is similar to that of columns.

Reinforced Concrete Jacketing

Reinforced concrete jacketing of a joint is performed in such a way that all
Fig. 6.15

1 - longitudinal bars
2 - horizontal ties
3 - vertical stirrups
4 - vertical stirrups
the members connected at the joint collaborate together. It is generally appropriate only when both columns and beams framing to the joint are also jacketed. For adequate bond between the original and the new concrete and possibly for welding of the new reinforcement to the existing reinforcement, the concrete cover must be chipped away. It is necessary that sufficient thickness of the jacket be provided in order that the great number of reinforcement bars (longitudinal column and beam bars, ties and stirrups) can be installed. Appropriate stirrups and closely spaced and adequately anchored ties are of great importance. The vertical and horizontal bars and ties must be assembled in a manner to form a space reinforcement. It, together with the concrete jacket, must be able to transmit all the internal joint forces. Sufficient jacket stiffness is also necessary.

In cases of damaged or poorly detailed joints, the reinforced concrete jacket can be located in the joint area only (Figure 6.15). The jacket encloses the column, the beam and the joint at all faces. Horizontal ties provide the necessary shear strength of the joint. Transversal vertical stirrups are connected with the horizontal ties. Note that the jacket projects above the top of the structural slab.

For heavily damaged structures interconnected jacketing of the beams, columns and joints is desirable (Figure 6.16). The column and beam additional reinforcement passes through the joint area. Additional horizontal and vertical ties in the joint area must be placed for providing adequate joint shear strength. Above and under the cross beam, horizontal column ties in the joint area must be closely spaced and spliced by overlapping. In case of deep cross beams, a connection with some horizontal ties passing through the beam web (i.e. through drilled holes) is advisable.

Depending on the forces acting in the area of an exterior joint (Figure 6.17) appropriate reinforcement should be provided in the joint jacket. Additional horizontal and vertical bars from the column and beam penetrate into the joint area. Closely spaced welded wire fabrics or reinforcing bars are placed at both of the joint faces. Welded wire fabrics in the joint area should be connected by welding to the original reinforcement or by attachment to the concrete with epoxy anchored dowels or power-driven nails. Additional stirrups or ties in the beam and column close to the joint improves the performance of this connection.

**Steel Plate Reinforcement**

Steel plate reinforcement is a technique which permits strengthening of joints without considerable changes in size. It can be used primarily in industrial type structures where heavy frames are located in one direction. Steel plates shaped according to the joint configuration are glued by epoxy resin to the concrete (Figure 6.18). The steel plates should be anchored to the joint by prestressed bolts. At uneven surfaces of the joint, a thin layer of expansive cement mortar for smoothing is advisable. Steel plate thickness must be at least 4 mm. Note that this technique is not practical when sizable beams frame into all four sides of a beam-column joint. The method will not provide reliable strengthening of the joint region for seismic resistance frames unless the steel plates are interconnected with similar steel plates or profiles used in strengthening the adjacent column and beam. External steel plate reinforcement should mainly be applied for strengthening procedures. Special measures must be taken into account for fire and corrosion protections.
1 - column reinforcement
2 - beam top reinforcement
3 - beam bottom reinforcement
4 - joint vertical stirrups
5 - beam stirrups
6 - column ties
7 - column ties in joint

Fig. 6.16
1 - column reinforcement; 2 - beam bottom reinforcement; 3 - beam top reinforcement; 4 - beam reinforcement; 5 - column ties; 6 - beam stirrups; 7 - welded wire fabric; T - tensile resultant

Fig. 6.17
Design Considerations

Joints should be designed for combined action of the internal forces (axial forces, bending, tension and shear) at the joint faces. For moment resisting "weak beam - strong column" frames, the plastic hinges occur in the beams close to the column faces. Therefore, beam yield forces act at the beam column interfaces (Figure 6.19) and the joint design calculations should verify that the joint provides adequate shear strength and sufficient beam reinforcement anchorage in the joint area. The development length of the beam bars anchored in the joint is determined from the critical section assumed to be the column core face. It has been observed that concrete outside of the confined core is inefficient for the reinforcement development.
6.1.5 Shear Walls

Shear walls, because of their great stiffness and lateral strength, provide the most significant part of the earthquake resistance of the building structure. Therefore, a severely damaged or a poorly designed shear wall must be repaired or strengthened in order that the structure's strength for seismic force can be significantly improved.

Local Repairs

Injections with epoxy resin for crack repairs in shear walls has become a fairly standard practice over the last decade. If neither bond deterioration nor concrete crushing have occurred, epoxy injections are capable of restoring approximately the original wall flexural and shear strength. However, the repaired wall will never achieve the stiffness of the original wall. This is due to the fact that not all of the small cracks can be epoxy injected. Another disadvantage is the rapid loss of strength of epoxy under fire action. The wide application of epoxy repair of walls is due to the procedures being simple, fast and economical without changing the original wall size and without evacuation of the inhabitants. Higher strength in walls than the original strength cannot be attained by epoxy injections. Therefore, if additional strength is required, another technique must be used. Epoxy injections should be performed according to the recommendations in Section 5.6.

Removal and replacement should be applied in cases of large cracks, partially crushed concrete and buckled reinforcement. After removing the loose concrete and roughening and cleaning the remaining surface, additional reinforcement or welded wire fabric should be placed at the repaired region. The choice of replacement materials (polymer mortar, cement mortar, concrete or shotcrete) depends on the degree of damage, the desired repair characteristics and the rehabilitation conditions. Nonshrinkage or expansive cement for mortar and concrete is often desirable. Special care must be paid to adequately compact the new concrete, especially at contact regions with the existing concrete.

Increase of Wall Size

Thickening the wall with reinforced concrete should be applied when the original strength of a damaged or undamaged wall is insufficient. There are different ways to add strength to an existing concrete shear wall (Figure 6.20). Shotcrete is a frequently used technique in strengthening concrete shear walls.

When shear strength is to be increased, thickening the web with additional reinforced concrete is necessary (Figure 6.20). Web reinforcement, consisting of horizontal and vertical bars is added depending on the strength increase desired. The reinforcement and the concrete must be anchored to the existing wall by appropriate roughening of the original concrete surface and by special anchor bolts. Appropriate solution is the application of epoxied bars with 90° hooks. Anchorage of the added web reinforcement can also be achieved by epoxying dowels in holes drilled in the boundary elements of the wall.

If an increase in flexural strength of a shear wall is required, reinforced concrete flanges must be added to both ends of the wall (Figure 6.20). The new flange concrete must be confined by appropriately detailed closely spaced hoops and cross ties. The anchorage of the new flange concrete to the original wall is very important. It can be performed by welding of special bent bar connectors to the new and the existing reinforcement or by epoxied anchor bars.
Fig. 6.20

1 - existing wall
2 - added wall
3 - added columns
4 - welding
5 - epoxied bar
If both flexural and shear strength must be increased, reinforced concrete must be added to both parts – the web and the cross section ends. Solutions with one-sided (Figure 6.20) or two-sided (Figure 6.20) web thickening are possible. The two-sided solution generally results in better behavior, however, it is more expensive and requires access to both sides of the wall. Ties through holes drilled in the existing wall and hooking around the reinforcing on each side provide a basketing effect which adds considerable strength to the wall beyond that which can be conventionally calculated.

An example of a two-sided shear wall strengthening is given in Figure 6.21.

1 - existing wall; 2 - existing slab; 3 - added longitudinal reinforcement; 4 - added wire fabric; 5 - diagonal connecting bars; 6 - added ties

Fig. 6. 21
Shear forces between the shear wall and the floor slab must be adequately transmitted. For this purpose, concrete dowel connections are made by opening holes through existing slab. These holes also serve for concreting the new wall parts beneath the slab. Diagonal reinforcement bars passing through the slab and anchored in the upper story and lower story walls provide an additional connection for shear transfer.

To provide adequate bond and connection between the original and new concrete and for possible welding of connection bars to the existing reinforcement, some concrete cover must be chipped away. The original wall surface should be thoroughly roughened and any paint or plaster removed. Additional shear is transmitted from the existing to the new wall by epoxied dowels in the existing surface.

Reinforced concrete thickening of shear walls should also conform to the following provisions:

- The strength of the new materials should not be less than those of the existing walls.

- The web thickness of the new material should be at least 5 cm. The thickness of the new shear wall flanges should be at least 10 cm.

- Both the horizontal web reinforcement area and the vertical web reinforcement area must be no less than 0.0025 times the gross area of the wall thickening.

- The area of the vertical reinforcement concentrated at the wall ends must be no less than 0.0025 times the gross area of the newly added cross section.

- The wall end ties diameter should be no less than 8 mm or 1/3 of the vertical end wall reinforcement bar diameter. The tie spacing should be no more than the thickness of the wall end thickening or maximum 15 cm.

- The new concrete should be anchored to the existing concrete with epoxied hooked dowels at a maximum of 60 cm in each direction and the existing wall surface should be thoroughly roughened.

**Design Considerations**

The strengthened shear wall should be proportioned depending on the effectiveness of the bond between the existing wall and the newly added thickening to resist the combined action of bending moments, axial force and shear. The transfer of the forces to the foundations must be provided adequately and may require extensive work. The increased stiffness of the strengthened element must be considered as it causes a redistribution of the lateral forces among the other shear walls in the structure.

6.1.6 Slabs

Slabs of floor structures primarily have to carry the vertical gravity loads. However, they must also provide diaphragm action and be compatible with all lateral resistant elements of the structure. Therefore, slabs must possess the necessary strength and stiffness. Damages in slabs generally occur in locations with irregularities, such as near large openings at concentrations of earthquake forces in slabs close to widely spaced shear walls and at staircase flight landings. Repair of slabs is necessary when damages occur. Strengthening is applied when there is insufficient slab strength or for increased strength in the region of newly-introduced shear walls.
Local Repairs

Injections should be applied for repair of cracks. Epoxy or cement grout can be used. Restoration of connection between the separated concrete parts can be achieved.

Removal and replacement procedures should be applied in cases of spalled concrete and broken or buckled reinforcement. Floor slabs or staircase slabs can be repaired in this manner (Figure 6.22). After the removal of the deteriorated materials new reinforcement is incorporated and it is welded to the existing reinforcement. Concrete with the same properties as the existing concrete should be used.

![Diagram](image)

1 - added reinforcement
2 - welding
3 - added concrete
4 - existing slab

Fig. 6.22

Increasing Slab Thickness

Strengthening by thickening of slabs should be applied in cases of insufficient strength or stiffness. The new materials are incorporated above or under the existing slab. In the first case (Figure 6.23.a), the flexural strength is increased because of the increased effective depth and ability to add negative reinforcement as supports. In the second case (Figure 6.23.b), the flexural strength increases because of the newly added tension reinforcement. In the case of 'a' normal concrete is used; in the case of 'b' the application of shotcrete is more suitable. The strengthening according to case 'a' provides greater floor slab stiffness necessary for the horizontal diaphragm action and is strongly recommended as the preferred method of the two. In case 'b', the behavior will be improved if the beams are also jacketed to increase continuity of the strengthening material.

For the compatibility of the existing slab and the newly-added reinforced concrete, obtaining excellent shear bond is of great importance. It can be achieved by means of the following:
1 - existing slab
2 - added reinforcement
3 - dowel
4 - anchoring bent bars
5 - welded connecting bars

Fig. 6.23

1 - existing slab; 2 - new slab; 3 - sand corner; 4 - epoxy glue;
5 - epoxied bolts; 6 - angle profile; 7 - anchor bolts or shoot
nails

Fig. 6.24
- Reinforced concrete lugs (Figure 6.24.a).
- Rough surface, realized by gluing of sand grains with epoxy resin (Figure 6.24.a).
- Epoxied steel dowel bars (Figure 6.24.b).
- Additionally shaped concrete lugs in slab voids (Figure 6.24.c).
- Steel dowels performed either by steel angle profiles anchored by power concrete nails or by epoxied bolts or wedge anchor bolts (Figure 6.24.d).

The roughening of the original concrete surface provides considerable improvement of the bond between the original and the newly added concrete. It can be performed by sandblasting or by chipping with special mechanical equipment.

**Strengthening Slab to Wall Connections**

An improved connection between the slab and a shear wall can be realized by reinforced concrete dowels (Figure 6.25). They incorporate reinforcement (longitudinal bars, stirrups and, if it is necessary, diagonal bars for additional shear transfer) into an elongated opening in the slab through the shear wall. The opening should be concreted with high-strength expansive concrete with special care for compaction to insure good bond.

![Fig. 6. 25](image)
6.1.7 Infilled Partition Walls

An infilled wall is a non-structural partition wall built of brick, concrete block masonry, hollow clay tile or stones held in place with mortar with a plaster finish on both sides. Although it is highly desirable in seismic regions to reinforce infilled walls and anchor them into the surrounding concrete frame members, infilled walls are frequently installed as unreinforced. Infilled partition walls in concrete framed buildings often sustain considerable damage in an earthquake as they are relatively stiff and resist lateral forces, often which they were not designed to resist, until they crack or fail. Damage may consist of small to large cracks, loose bricks or blocks or an infill leaning sideways. Damage may also result in the concrete frame members and joints which surround the infilled wall.

Repair and strengthening techniques obviously depend on the degree of damage and the decision to strengthen the infilled wall. Repair of an infilled wall without strengthening is generally a non-structural matter with loose or broken bricks or blocks being replaced and plaster patched or renewed. Infilled walls repaired in this manner obviously cannot be relied on for lateral bracing in future earthquakes as both their strength and stiffness will be less than their original undamaged condition.

Strengthening of infilled walls can often be performed at a small increase in cost above simple repairs and, thus, provide a significant increase in lateral strength of smaller concrete structures. Depending on the amount of strength desired from the strengthening procedure, different details can be utilized to achieve the strength. Loose or broken bricks or blocks are first replaced and loose mortar between units is replaced. A light wire mesh or welded wire fabric is then installed on both sides of the infilled wall, with heavy wire ties passing through holes drilled in the wall at reasonable intervals hooking the two layers of fabric together. This method is similar to the reinforced concrete jacketing of masonry walls discussed in Section 8.4 and illustrated in Figure 8.15, although the entire infilled wall should be strengthened. A cement plaster finish about 2 cm thick or a shotcrete layer of concrete about 4 to 6 cm thick is then installed on both sides of the infilled wall. This reinforced layer at both faces then serves to strengthen the wall and provide a basketing effect to contain the infill units and force the wall to act as a unit. Considerable additional strength can also be obtained if the reinforcing is connected to the concrete frame at the perimeter of the infilled wall, either by epoxy dowels, expansion anchors or power-driven concrete nails.

The effect of strengthening an infilled wall must be considered by analysis on the surrounding elements of the structure. Infilled walls are extremely stiff and effective in resisting lateral forces, but all forces must be transferred through the concrete elements surrounding the infilled walls. These members, particularly the columns and beam-column joints, should be checked for shear strength. Furthermore, columns and foundations should be checked for capacity to resist overturning forces resulting from the shear resistance of the strengthened infilled wall.

6.2 Introduction of New Structural Elements

6.2.1 General

The lateral force capacity of an existing structure may be increased by adding new structural elements to resist part or all of the seismic forces of the structure, leaving the old structure to resist only that part of the seismic action for which it is judged reliable.
The newly added elements may be:

- Shear walls in a frame or skeleton structure.
- Additional shear walls in a shear wall structure.
- Additional frames in a frame or skeleton structure.
- Bracing (steel or reinforced concrete) in a frame or skeleton structure.

The choice of the type, number and size of the added elements depends on the particularities of the existing structure and the functional layout of the building. Therefore, only some outline principles regarding the structural role, the location, the structural analysis and the dimensioning of these elements, will be given in this section.

In every case, the new structure, composed of the old structure and the additional members, has to be analyzed and designed as such. There are two possible ways of shaping such a structure. Elements can be introduced whose stiffness for lateral forces is of the same order of magnitude as the old structure. In this case, the lateral force is taken over jointly by the old and the new structure depending on their relative rigidity and location within the structure. Shear walls can also be added with a stiffness considerably higher than that of the old structure. This is feasible for buildings where the interior space is already partitioned and existing partitions could be replaced by earthquake resisting shear walls, or new walls could be added with no significant disturbance of the functional layout.

In such cases, the new shear walls should have sufficient strength and stiffness to provide the entire lateral force resistance. The existing structure must be compatible with the strengthening elements and able to deform without failure in future earthquakes.

Incorporation of new structural components in an existing building will change the dynamic behavior of the whole space structure considerably during an earthquake. The increase in stiffness will also tend to increase the seismic design forces for most typical structures. It also causes considerable redistribution of the lateral forces between the earthquake resisting elements. Therefore, it is very important that the most favorable conditions be created whenever possible, as follows:

- Avoiding large concentration of forces in members with small strength and/or ductility capacities by locating the strengthening elements uniformly throughout the structure.
- Improving the distribution of lateral force by reducing the effects of torsion and irregularities.
- Providing sufficient strength, stiffness and ductility of the individual elements and of the whole structure.
- Providing adequate strength in connections between the existing structure and the newly added elements.
- Providing stiffness compatibility of the existing and the newly added elements.

Past earthquakes have demonstrated that buildings with irregular configurations have suffered extensive damage. Therefore, it is often
necessary to introduce added structural elements to the existing structure to provide regularity in plan and elevation. The new earthquake resistant elements should be placed in such a manner as to minimize the distance from the center of mass to the center of rigidity. Also, temperature, shrinkage and creep influences which can produce significant redistribution in the internal forces between the existing and the new structural elements must be taken into account.

Some examples of favorable or unfavorable distribution of added structural elements are shown in Figure 6.26. In the case of a pure skeleton structure with insufficient lateral resistance, shear walls should be added in architecturally convenient places. The distance between the shear walls must be less in the case of flexible floor structures (i.e. poorly connected prefabricated panels) (Figure 6.26.a) and can be greater for stiff monolithic reinforced concrete slabs (Figure 6.26.b). In long, rectangular buildings the shear walls placed at the corners and orientated along the longitudinal direction (Figures 6.26.c and 6.26.d) limit the deformation due to temperature changes and can produce considerable additional internal forces in the structure. Therefore, it is desirable that the shear walls be orientated in transverse direction at building ends as in Figures 6.26.a and 6.26.b. An example of poor shear wall distribution is shown in Figure 6.26.d, where the longitudinally orientated shear walls restrict the free deformations, produce large eccentricities of the resultant seismic forces and the lever arm of the transverse walls is not sufficient for carrying the torsion.

The same considerations are valid in the case of buildings with an insufficient number of shear walls. It is necessary to locate additional walls with sufficient strength while reducing unfavorable torsional effects (Figure 6.27). Vertical irregularities of shear walls resulting in soft stories or short, stiff columns must also be eliminated by providing walls with consistent strength and stiffness over their full height.
6.2.2 Shear Walls

The addition of new reinforced concrete shear walls are undoubtedly the best method of strengthening an existing structure for improved seismic performance. The walls are generally cast-in-situ but may be installed with shotcrete. Precast, prefabricated concrete elements can be used to form new shear walls but their details are extremely critical and difficult to provide the desired performance.

Monolithic reinforced concrete shear walls can be situated either along the periphery of the building (Figure 6.28) or inside of it (Figure 6.29). Adding walls along the periphery is often easier as it does not upset the interior function of the room layout, although it will alter appearance and window layouts. In this first case (Figure 6.28), the main vertical end reinforcement for flexure and the web reinforcement for shear passes continuously along the entire height of the wall. The main problems are providing an adequate connection of the new shear walls with the floor and roof diaphragms and the foundations. This connection must transmit the shear forces between the existing floor or roof structure and the new shear wall. The connection can be performed by ties anchored into the floor structure, diagonally placed in plan or beams formed from the shear walls and anchored into the floor structure by additional cast-in-situ reinforced slab. The additional slab is anchored by incorporated reinforced dowels into the existing floor structure.

Monolithic shear walls placed inside of the building (Figure 6.29) are connected with the floor structures by vertical longitudinal reinforcement passing through opened holes in the existing slab, concrete dowels or lugs formed by opened holes in the slab, or diagonal bars placed in the dowels. The main longitudinal reinforcement situated at the wall ends passes continuously through the holes in the slab and must be spliced at convenient locations. However, most of the web reinforcement is interrupted at every slab level and it must be spliced by diagonal and vertical bars placed in the holes through the slab. The holes through the slab must be large enough to transmit the shear forces as well as place the concrete for the wall below the slab. Detailing attention must be paid to achieve adequate and effective confinement of the shear wall ends. Enlargement of the cross section wall ends by flanges is advisable. In order to improve ductility and prevent buckling of the main longitudinal reinforcement, closely spaced ties should be placed in this end region. The ends of the shear walls are also generally attached to existing
building columns to provide gravity loads to counter uplift tendencies of the wall ends (Figure 6.29).

Figures 6.30 and 6.31 give additional details of added shear walls in buildings.

Precast, prefabricated units can also be utilized to form new reinforced concrete shear walls. The great difficulty with such units is developing economical details that provide connections with adequate strength and ductility. Welded details frequently result in large eccentricities within the connections or a brittle condition when the highest stresses exist in the welds. For this reason, welded connections are generally not desirable unless stresses are very low and anchors are well embedded in substantial concrete. The most desirable connections for precast concrete units forming a new shear wall utilize cast-in-situ closure strips between all elements with reinforcement extending from the precast unit and being well anchored in the closure strip. Erection problems may also exist for precast, prefabricated concrete elements as cranes cannot reach inside an existing building. Thus, the erection considerations alone may prohibit using precast element for new shear walls.
Fig. 6.29

1 - added shear wall
2 - existing structure
3 - added concrete
4 - added reinforcement
5 - added ties
6 - dowel
7 - added diagonal bars
8 - added vertical bars
Additional vertical reinforcement

Additional horizontal reinforcement

Fig. 6.30

Fig. 6.31
Infilled walls consist of reinforced concrete or masonry located in the plane of existing columns and beams, as shown in Figure 6.32. Structurally, the infilled wall is a shear wall consisting of the existing frame working together with the infill. The beams and columns act as tension or compression edge members. The columns also must resist high shear forces as they transmit the horizontal forces from the infilled wall to the floor system. This may, in some cases, completely invert the initial design assumptions, for instance, a column designed for gravitational loads and assumed as axially or eccentrically compressed, may become tensioned under seismic loads. It is essential that the existing frame members have sufficient strength for carrying the additional forces caused by the infilling.

Infilled walls are an extremely important method for strengthening existing concrete frames of one to three stories in height. The infilled wall can form interior partitions, particularly between rooms or units of residential construction. The infilled panel can be of cast-in-situ concrete or shotcrete connected to beams and columns, as shown in Figure 6.32.a. This solution requires adequate strength in the existing columns for the resulting overturning forces from the infilled shear wall. The existing column should also have adequate shear capacity as high shear stresses will result if movement occurs in the connection provided between the new infilled panel and the beam. Cast-in-place infilled walls can also be connected only to the beams as shown in Figure 6.32.b. This type of infilling is appropriate when the existing columns will be overstressed and new columns or boundary elements must be provided at the ends of the infilled panels. An enlarged new boundary element can be provided, as shown in Figure 6.33.c, which allows tensile reinforcing to pass through the floor slab on either side of the existing floor beam.

Infilled panels can also be constructed of reinforced masonry of various types. Brick or concrete block masonry panels can be reinforced and attached to the concrete frame at the perimeter of the panel in the same manner as cast-in-situ concrete. Precast concrete panels can also be used for infilled walls, but their connections at their edges must be carefully conceived to provide sufficient strength and ductility. Precast panels may also be difficult to install inside an existing structure if the floor is unable to support the weight of the precast concrete panel with its lifting equipment.

The most important part of constructing infill walls for seismic strengthening is properly detailing the connections. Connections with the column can be detailed by the following methods:

- Chipped keys in the existing columns. Additional anchor bars are welded to the longitudinal column reinforcement (Figure 6.33.a).

- Chipped keys in the existing columns. Diagonal anchor bars are welded to the longitudinal column reinforcement and additional ties are welded to the existing column ties (Figure 6.33.b).

- Chipped keys in the existing column plus steel dowels epoxied in the existing column (Figure 6.34.a). Additional longitudinal reinforcement can be added depending on conditions at the floor beams (Figure 6.34.b).

- Encasement or jacketing of existing column if the column must be strengthened for compression (Figure 6.34.c).

Connections with beams can be detailed by the following methods:
a - cast-in-place infilled shear wall
b - cast-in-place infilled shear wall separated from columns

Fig. 6.32
1 - added infilled wall; 2 - existing beam; 3 - existing columns; 4 - steel spiral; 5 - exoixed bolt; 6 - concrete dowel; 7 - welded anchor bar; 8 - diagonal anchor bar; 9 - added welded stirrups; 10 - added welded ties; 11 - groove; 12 - prestressed tendons; 13 - wedge anchor bolt; 14 - steel dowel; 15 - epoxy glue; 16 - embedded steel plates; 17 - welding; 18 - added diagonal bars; 19 - added horizontal bars

Fig. 6.33
Possible dowel

R.C. infill wall

a) if sufficient reinforcement in the existing column

b) if supplementary vertical reinforcement has to be added

c) if the column has to be strengthened for compression

Fig. 6.34
- Epoxied bolts in predrilled holes into the beams (Figure 6.35.a).
- Combination of chipped keys on the existing beam with additional anchor bars, welded to the existing reinforcement and epoxied bolts in predrilled holes (Figure 6.35.b).
- Combination of chipped keys on the existing beam with additional diagonal anchor bars welded to the existing longitudinal reinforcement and additional stirrups welded to the existing ones (Figure 6.35.c).
- Steel dowels constructed by welding together steel plates attached to the existing beam by epoxy glue and wedge anchor bolts (Figure 6.35.d).

![Fig. 6.35](image-url)
Several important considerations when detailing infilled walls are as follows. The embedded length of the wedge anchor bolt should be equal to 5\(d\) or more and should extend well beyond the longitudinal reinforcement of the existing beam or column. Sufficient reinforcement against splitting should be arranged around the dowel reinforcement in the infill wall, either by spirals as shown in Figure 6.35.a or hairpin-shaped bars cast in the infill. The ratio of the length to depth of a key is approximately 5:1. The key length should be no less than 15 cm.

Design calculations should determine that adequate strength is provided not only for the infilled wall but especially for the connections between the infilled wall and the existing frame. Additional calculations must verify that adequate strength of the existing frame (especially the columns) is present under the new loading conditions resulting from the infilled walls acting as shear walls.

6.2.4 Wing Walls

The lateral strength of existing columns can be increased by this technique which involves adding wall segments or wings on each side of an existing column. The wing wall generally has a thickness considerably less than the width of the adjacent column. Wing walls are generally placed symmetrically about the existing column or can be partially infilled in the existing frame and either attaching or encasing the existing column. These general types of wing walls are illustrated in Figure 6.36.
The most important aspect of designing wing walls to strengthen a structure is providing adequate connections to the existing structure, which will be highly stressed in this technique. Connection details as previously discussed under jacketing and infilling are appropriate for wing walls, with emphasis being given to the best of those details.

The wing walls should be anchored at all beams and at the foundation level in a beam extending between two neighboring footings in order to fix the wall at this level and prevent overturning (Figure 6.37). It is also most desirable to provide continuity of the vertical reinforcing at the ends of the wing wall, as shown in Figure 6.37. This continuous reinforcement combined with adequate shear connections along the beams and column and a base fixity create a vertical cantilever wall system which can effectively resist earthquake forces.

The new wall system comprised of wing walls will shorten the clear span of the beams where the wing walls are attached. There will be interaction between the new wing walls and existing beam creating large reversible moments in the beam at the face of the wing wall and the beam may not be detailed for reversible moments at that location. This may require also strengthening the beam or making other provisions to ensure that the beam does not fail. In some structures where the beam may be a deep exterior beam, the beam can be strengthened if necessary and designed to function with the wing walls as a large wall with regular openings.
If the continuity of the vertical edge reinforcement on the whole height of the wall cannot be realized, the state of stress is similar to that of Figure 6.38. The existing column will act as a tension member and must be checked as such. Shear capacity along the vertical joint between wing wall and column must be ensured either through appropriate shear connections or by the shear capacity of the existing beams at the junction with the columns. At the same time, shear and bending in the existing beams at the wall extremity is large and the necessary capacity must be available in these beams at each floor level. Adequate attachment of the wing wall to the beams is also extremely important in this case.
6.2.5 Trusses and Diagonal Braces

When a large number of openings are required, vertical trusses or diagonal bracing can be added as strengthening measures. If the strength capacity of the existing columns is insufficient, which is usually the case, a full vertical truss with vertical chords can be added. If columns and beams have sufficient strength and ductility, the steel diagonal braces can be added within the concrete frame to form a vertical truss of existing beams and columns and new diagonals.

New vertical trusses can be constructed with steel members, cast-in-situ reinforced concrete members or a combination of the two. If reinforced concrete members are used, all members should be confined with closely spaced ties for the full length of all members to provide adequate member ductility. Figure 6.39 illustrates one version of a structural steel truss added to a multistory building. Obviously, many types of trusses are possible, depending on the existing frame geometry and the need for openings at particular locations.

Truss details must be designed to minimize or control eccentricities and provide adequate connections to the existing structure. Such connections are difficult to install with adequate strength and designers must give particular attention to the details to be used. Combined systems of structural steel and reinforced concrete members are quite difficult as connections should be sufficient to develop the strength of the members. Connections to the floor and roof diaphragms can be achieved by reinforced concrete dowels formed in enlarged holes in the slab, steel dowels placed in holes in the slab and welded to the steel truss profiles, encasement of horizontal truss members in new concrete anchored to the floor slab or concrete slab holes opened for passing of vertical truss columns. Connection of vertical truss chords to existing columns to mobilize gravity loads to counter uplift tendencies can be achieved by encasement of the existing column and the new truss chord in a single concrete jacket or by appropriate welding to the existing longitudinal column bars and the new truss chord. It is necessary that the steel truss be proportioned to be compatible with the stiffness of the existing building and that special measures for corrosion and fire protection be taken.

Steel diagonal braces can be added to existing concrete frames to form a vertical truss in cases where sufficient beam and column strengths (especially shear strength) and ductilities exist. Braces should be arranged so that their center line passes through the centers of the beam-column joints. One or two diagonal braces can be installed depending on the geometry and other considerations. Angle or channel steel profiles are generally used. The critical detail is providing proper attachment of the brace to the frame structure joints. Possible connections include steel collars fabricated of steel plate above and below the joint and welded and grouted in place or a complete steel box fabricated of steel plates and angles when lateral beams do not complicate the existing beam-column joint area. Steel bracing produces additional forces in the existing structure (column, beams and joints). The shear strength in the columns close to the joints must be carefully investigated, especially close to the new steel collars.

6.2.6 Foundations

Repair and strengthening of foundations is a difficult and expensive construction procedure. It should be performed in the following cases:

- Excessive settlement of the foundations due to poor soil conditions.
1 - added steel truss
2 - existing structure
3 - steel dowel
4 - horizontal steel rod
5 - diagonal steel rod
6 - steel joint plate
7 - added concrete

Fig. 6.39
- Damage in the foundation structure caused by seismic overloading.
- Increasing of the dead load as a result of the strengthening operations.
- Increasing the seismic loading due to changes in code provisions or the strengthening operations.
- Necessity of additional foundation structure for added floors.

1 - existing foundation; 2 - existing column; 3 - reinforced jacket; 4 - added concrete; 5 - added reinforcement

Fig. 6.40
Repair and strengthening of foundation structures can be performed by strengthening the existing foundation structure, adding new foundation structure, or modifying the soil for improved foundation support.

Strengthening of existing footings necessitates increasing the dimensions of the footing to increase the bearing area of footing. The additional soil pressure must be equalized in the footing body. It is easy to visualize in the case of columns strengthened with jacketing (Figure 6.40). The footing belt, arranged at the bottom, is of great importance for transmitting the inclined forces (in the strengthened upper part) to the soil. Therefore, strong belt reinforcement with adequate splicing is necessary.

In the case of an unstrengthened column, soil pressure applied to the strengthened part must be transmitted directly to the existing foundation body (Figure 6.41). This can be achieved by incorporating a steel profile under the ends of the existing footing. Adequate reinforcement must be placed in the newly added footing body.

Fig. 6.41
Introduction of new foundations is necessary when new shear walls or similar elements are constructed in the building structure (Figures 6.42 and 6.43). Special attention should be paid to incorporating the existing footings into the newly introduced foundation structure in such a manner that the parts will function together properly.

1 - existing columns; 2 - existing foundation; 3 - added infilled shear wall; 4 - added reinforcement concrete; 5 - added reinforcement; 6 - welded steel plates

Fig. 6.42
The anchorage of the main vertical shear wall end reinforcement into the foundations is very important. This can be accomplished either by anchorage of the reinforcement in predrilled holes in the existing foundation with epoxy resin or by inclined reinforcement bars in the newly constructed foundation parts. The inclination of the reinforcement bars can be accomplished by welding to steel plates (Figure 6.43.b) with adequate ties to resist any unbalance in lateral components of force. The axial forces from the overturning moment must be resisted by adequately detailed reinforcement placed in the newly constructed foundations. The wall shear forces must be transmitted to the foundations by sufficiently anchored web reinforcement.

Fig. 6.43

1 - existing columns; 2 - existing foundations; 3 - added cast-in-place infilled shear wall; 4 - added concrete; 5 - added reinforcement; 6 - diagonal anchor bars; 7 - welding; 8 - steel plate
Modifying the soil for improved foundation support can be performed by compaction grouting in case of fine grained soils, or by chemical solidification in case of relative permeable granular material. Compaction grouting results in densification of the soil and, therefore, it reduces the potential for liquefaction. Providing cohesion in granular soils by chemical solidification also reduces the potential for liquefaction.

New grade beams must sometimes be added to a strengthened structure either to support added loads or provide a positive tie between the footings. Figure 6.44 illustrates one method of installing a new grade beam incorporated with jacketing of the columns. The grade beam should have continuous reinforcement and be well connected to the existing footings.
7. REINFORCED PREFABRICATED STRUCTURAL SYSTEMS

This section discusses seismic repair and strengthening procedures appropriate for reinforced prefabricated structural systems in common usage in the Balkan countries. The principle applied to other structural systems, such as cast-in-situ reinforced concrete structures, also apply to these prefabricated systems although the details of repair will be somewhat different. In these systems, the connections are generally the weakest part of the structure for seismic loading and the repair and/or strengthening details require particular attention to those connections. This section is not intended to cover all prefabricated systems being constructed in the Balkan countries, but only to give typical examples which can be extended to other systems.

7.1 Large Panel Structural Systems

7.1.1 Introduction

Prefabricated large panel systems consist of large concrete wall and floor panels which are connected in the horizontal and vertical directions with joints. The horizontal floor and roof panels can be either one-way spanning prefabricated slab elements or two-way spanning elements between surrounding walls. Prefabricated wall panels are typically one-story high and both horizontal and vertical joints usually exist between the panels. The basic configuration of large panel buildings can be a cross-wall system, a long-wall system or a two-way system.

In the cross-wall system, the load bearing walls are placed perpendicular to the longitudinal axis of the building and provide resistance to seismic loads in their direction and support the gravity load from one-way spanning floors. Walls in the opposite direction provide resistance to seismic loads in that direction. In the long-wall system, the load bearing walls are placed parallel to the longitudinal axis of the building and provide resistance to seismic loads in their direction and support the gravity loads from floor elements, while non-load bearing walls are placed perpendicular to the longitudinal axis of the building and provide resistance to seismic loads in that direction. The bearing walls in the two-way system are placed in both directions and provide vertical and seismic load resistance. Closely related to the two-way large panel system is the box system where concrete boxes are cast as integral units and assembled with connections.

A wide range of joints between panels is possible and their type and composition depends on the construction procedures. In general, joints may either be "wet" or "dry". Wet joints are constructed with cast-in-situ concrete in the joint region between prefabricated panels. The structural continuity through the joints is obtained by reinforcing bars which resist shear and tensile forces. Dry joints are constructed by welding or bolting together steel plates or other steel inserts which are cast into the prefabricated panels.

Repair procedure of large panel prefabricated structures after a damaging earthquake depends on the configuration and type of the large panel connections, as well as on the type and damage level. Depending on the desired earthquake resistance, the level of damage, the type of system and connection, the structure can be repaired and/or strengthened in many different ways.
7.1.2 Repair of Large Panel Prefabricated Construction

Cracks typically appear in the connections between the prefabricated panels of large panel system and the cast-in-situ concrete when wet joints were used. These cracks may or may not develop with a local crushing of the concrete. In structures with dry joints, cracking and spalling may occur at the joint locations and inserts may fail or be distressed.

Horizontal wet joints occasionally develop cracks in connections between prefabricated and cast-in-situ concrete as shown in Figure 7.1. These cracks may be larger and often occur along the entire length of the connection occasionally with some sliding. In some cases, it is possible to observe local crushing in concrete in the end part of the panel in the cast-in-situ connection as well as in the part of the prefabricated panel due to high compression from the gravity and seismic overturning forces. To repair these joints, the joint is first cleaned with compressed air under high pressure in order to remove all dry particles deposited due to slipping and crushing of the concrete. At the places of local crushing, all loose concrete should be removed and new concrete or epoxy mortar installed. The remaining part of the horizontal joints is then grouted with adequate emulsion (cement, epoxy, etc.).

Figure 7.2 presents a typical vertical connection of a large panel system with cast-in-situ joints of two types: with slight keys or lugs in the panel edges (Figure 7.2a) and with large keys or lugs (Figure 7.2b) along the contact of the prefabricated and the cast-in-situ concrete. The characteristic damage to the vertical connections is caused by developing cracks along the contact zone and by partial crushing of the keys or lugs. In case of fine cracks without local crushing of concrete, the cracks should be cleaned and grouted over the whole height applying adequate grouting emulsion to fill the cracks. In the case of larger cracks with local crushing of the keys or lugs, it is necessary to remove the crushed concrete and a part of the monolithic concrete and new concrete or grout should be placed using forms as required.

Dry joints consisting of welded or bolted plates or inserts generally have cracked or crushed concrete adjacent to the connections and occasionally bent steel connectors, sheared bolts or cracked welds. Repairs to the concrete generally consist of removing the loose or crushed concrete and replacing it...
with cement or epoxy grout and filling cracks with epoxy injection. Distress to the steel elements of the connections are repaired by suitable welding, replacement or adding new steel pieces. If the connection has been extensively damaged, it may be more suitable to strengthen the connections with new connections rather than spending the effort to repair the badly damaged connections.

Damage occasionally occurs within the large panels away from the connections, especially adjacent to door or window openings or at discontinuities in a floor system. Repairs of these cracks or other damage is similar to repairs in cast-in-situ structures, utilizing epoxy injection or other techniques as appropriate.

Fig. 7.2 Vertical connection in large panel system

a. with small keys
b. with large keys
7.1.3 Strengthening of Large Panel Construction

Strengthening of large panel prefabricated structures can involve many different techniques depending on the degree of damage or lack of strength in one particular direction. If there are sufficient walls in both directions, then strengthening may consist of making the existing panel joints stronger or modifying them for improved performance. If one direction of the building has insufficient walls, which may occur in buildings with one-way floor panels and minimal walls perpendicular to the bearing walls, then new walls can be added and/or existing walls strengthened.

Strengthening of existing panel joints usually consists of adding steel angles parallel to the joint with bolts extending through the panel in drilled holes or set in epoxy in drilled holes (Figure 7.3). The panel joint is first repaired by usual techniques. The bolted angles can be installed to the length desired to achieve sufficient strength with bolts spaced about 10 diameters apart. The legs of the angles should be long enough to allow the bolts to be placed away from the edge of the panel so that the full strength of the bolts can be achieved and premature failure will not occur due to cracking or spalling at the edge of the prefabricated panel.

Figure 7.4 presents a characteristic large panel structure with longitudinal bearing walls and minimal transverse shear walls for stiffening. Considering the insufficient stiffness and resistance in the transverse direction, it has been decided to add new transverse shear walls over the entire height of the building including new foundations beneath the walls. Furthermore, considering the damage level of the building, it has been decided to strengthen the central stairway core by adding new reinforced cast-in-place walls to strengthen the existing ones. Strengthening of the panel walls (Figure 7.5) is carried out by construction of reinforced concrete columns at the vertical cast-in-place connections, thus simultaneously connecting the prefabricated panels and the newly constructed walls in which their horizontal reinforcement is anchored. The vertical reinforcement over the height of the walls is continued at the floor structure level through holes drilled in the floor slab. The contact area of the panel and the new concrete is chipped and keyed for improved bond. Reinforcement bars are placed and the new layer of concrete is either poured with forms as necessary or by the shotcrete method.
When walls are added analysis must be performed to evaluate the redistribution of the horizontal seismic forces to the various wall elements. Some walls may become stiffer due to flanges at their ends and floor connections and/or foundations may also require strengthening.

7.2 Prefabricated Frame Structural Systems

7.2.1 Introduction

Prefabricated frame structural systems are used in industrial buildings as well as in multi-story housing buildings. Prefabricated frame systems consist of linear beam or truss and column elements which are jointed in a spatial structure. The basic features which differentiate different systems of this type is the type of joints and the type of bracing provided to resist lateral forces.
There are several types of frame element joints. Rigid beam-column joints are constructed to transmit the vertical and gravitational loads as well as the seismic forces by frame action. This type of joint is used in some multi-story building structures. Joint continuity is sometimes achieved with post-tensioning tendons. Frame structures of this type seldom have adequate detailing of reinforcement for ductile performance. Hinged beam-column joints are designed as non-moment resisting joints of linear elements and are typically used in industrial halls and buildings. This hinged connection is sometimes located away from the column at the location of zero moment of a continuous frame.

Lateral forces of the existing structure may be resisted by a system of shear walls or diagonal bracing or by the prefabricated frames themselves. In strengthening buildings of this type, it is generally necessary to add shear walls or diagonal bracing to make the structure more rigid and to prevent the prefabricated frame structure from being the primary source of lateral force resistance. In some cases, it may be possible to jacket the existing frame members to achieve suitable strength and ductility for seismic force resistance.

7.2.2 Repair and Strengthening of Prefabricated Frame Systems

The characteristic damage sustained by this type of structure and its non-structural infill elements are caused by the flexibility of the basic system. Structural damage usually consist of cracking and distress to the joints and possible failure of members which were not detailed for sufficient ductility. Infill elements generally sustain considerable damage due to their high relative stiffness compared to the structural frame and their brittle characteristics.

The repair of this type of building consists of repair and/or strengthening of the damaged joints and members. Damaged members are repaired by techniques similar to those used for cast-in-situ concrete members. Joints are repaired by removing loose concrete, injection of cracks with epoxy or grout and patching spalled areas with a suitable grout. Repair of members and joints may actually consist of strengthening those areas by jacketing or other techniques to provide confinement or a new method to resist the appropriate forces. Prefabricated industrial frame buildings generally have damage to bolted or hinged connections which may require repairs consisting of added bolts or joint reconstruction in addition to repairs of cracking and spalling.

Strengthening of multi-story prefabricated frame buildings is generally performed by adding new reinforced concrete shear walls in both orthogonal directions over the entire height of the building. In this case, the shear walls are designed to resist essentially all of the seismic loads, so that the prefabricated frames carry only the vertical loads plus that portion of the seismic loads proportional to their rigidity relative to the added shear walls. This strengthening solution usually allows the prefabricated joints to be repaired to the same capacity as before the damage.

With systems utilizing prestressed joints or members, prior to the repair and strengthening, it is necessary to check if the prestressing tendons have lost any tension. If there is no loss of tension in the tendons, the most effective repair procedure is to fill in the cracks and joints with an appropriate emulsion. When joints have become loose due to excessive strain in the prestressing tendons or due to anchorage failure, it is necessary to install new tendons, cast concrete as appropriate and post-tension the tendons. New shear walls would be installed in both orthogonal directions to resist the seismic loads in these structures.
Strengthening of prefabricated frame industrial buildings is often difficult as shear walls can often be added only in the exterior frames of the building and the roof may contain skylights or other discontinuities which preclude sufficient diaphragm action to span between shear walls at the building's perimeter. If interior shear walls can be added parallel to the frames and not disrupt the functional use of the building, such a solution is the preferred strengthening solution. When shear walls cannot be added, then it is necessary to add exterior buttress walls or strengthen the existing column and foundation (Figure 7.6) to provide sufficient strength, stability and ductility to cantilever from the ground and resist seismic forces. When designing this type of strengthening care should be taken not to make a sudden transition of the strength or stiffness along the height of the column.

For strengthening prefabricated industrial frame structures in the longitudinal direction, it is possible to add new reinforced concrete shear walls or to all new rigid portal frames designed like a shear wall with an opening or adding a new diagonal bracing system of structural steel, as shown in Figure 7.7. When adding steel diagonal bracing, it may also be necessary to add horizontal and vertical steel members anchored to the existing concrete with epoxied bolts to transfer forces between the concrete structure and the steel bracing and to strengthen the columns for seismic overturning forces. The diagonal bracing may also be constructed of reinforced concrete.
7.3 Lift Slab Systems

Buildings constructed according to the Lift Slab Systems method of construction consist of reinforced concrete flat plates, which are cast in stacks at ground level and then lifted into place with jacks on each column. In addition to the lift slabs, structures of this type include columns and lateral resisting structural elements, such as shear walls, coupled shear walls, staircase cores or frames. A column-slab joint is detailed as a hinged joint. Thus, the columns resist axial forces only. The shear walls are dispersed along the contour of the building or between the columns. The reinforced concrete staircase core is connected to the flat plate. The foundations under the shear walls and the columns are connected together.

The damage typically observed in these types of structures which has caused a decrease of the structural capacity are: crushing of the concrete; buckling of the reinforcement, shear cracking of the reinforced concrete and brick shear walls; cracking of the slabs due to bending; disturbance of the connections between the columns and the slabs, as well as those between the slabs and the vertical strengthening connections; and settlement of the soil under the foundations. The measures for repair and strengthening of damaged structures built according to the Lift Slab method can be classified into two groups:

- Measures for increasing the total stability of the building against seismic forces.

- Measures for repair and increasing the strength of separate structural elements.
The introduction of new reinforced concrete shear walls is the strengthening solution most often applied. This method is applied in the case of both insufficient earthquake building resistance and when structural elements are cracked and deformed. The newly introduced shear walls must be appropriately dispersed in order to avoid significant torsion. An example of a new shear wall detailing is given in Figure 7.8. The connections between the new shear walls and the existing structure are of great importance. They can be detailed by providing dowels reinforced with diagonal bars through holes opened in the slab and by connecting reinforcement ties, welded to the longitudinal reinforcement of the existing column and anchored in the new shear wall.

1 - shear wall; 2 - reinforcement; 3 - dowel bars; 4 - connection reinforcement; 5 - existing columns; 6 - dowel openings; 7 - weldings

Fig. 7.8
The forces caused by seismic bending moments are transmitted by shear wall end longitudinal reinforcement bars to the foundation. This reinforcement passes continuously through the opened holes in the slab. The newly introduced shear wall foundation is a wall footing connected with the existing individual footings. The vertical end shear wall reinforcement is anchored into the existing footings in predrilled holes with the bar set in cement or epoxy mortar (Figure 7.9).

The repair or strengthening of the separate structural elements depends on the type and degree of damage. For example, crack repair of a shear wall might involve chipping away the concrete cover for a width about 60 cm in the cracked region, placing welded wire fabrics (diameter of the bars equal to 6 mm) in the free space and welding to the existing reinforcement and restoring the chipped concrete by shotcrete. Repair of cracks of widths up to 1 mm is performed by epoxy or cement compound injections. Large cracked lintels or shear wall crushed concrete zones are replaced by removing and replacement by new materials. Additional reinforcement is also added in heavily damaged regions. This reinforcement is welded to the existing reinforcement. Disturbed connections between the existing floor slab and the shear walls are repaired by introducing new reinforced concrete dowels (Figure 7.10) or, even better, also anchoring steel bars with epoxy mortar in holes drilled in the existing concrete on both sides of the joint.

1 - existing columns; 2 - existing column bases; 3 - new shear wall; 4 - new shear wall base; 5 - anchorage bars

Fig. 7.9
1 - floor; 2 - staircase walls; 3 - dowels; a - distance between dowels
8. MASONRY STRUCTURES

8.1 General

The repair and strengthening of stone and brick masonry structures uses many of the same techniques and methods discussed earlier in this document. Masonry structures are bearing wall systems, and earthquake damages can generally be traced to one or more of the following conditions, namely, insufficient stiffness and strength of roof and floor diaphragms, the absence of reinforced concrete tie beams or belts, inadequate ties between the exterior walls and the floor and roof systems allowing the wall to fail perpendicular to its plane, and insufficient strength and instability of the walls. Particularly for stone masonry structures, damages may also occur due to an insufficient interlocking at wall intersections.

Emergency measures may typically involve providing vertical supports and/or lateral wall bracing to prevent imminent collapse. See Section 3.3. Temporary support may also be required for balconies and cornices which are fixed into a damaged wall, where the overlying wall was acting as a counterweight.

Repair of cracks or replacement of damaged wall sections are essential features of the repair process unless the strengthening procedures selected resist all loads and forces to be carried by the wall.

In general, strengthening schemes for masonry structures must insure that the floor and roof diaphragms and the walls are well interconnected to preclude failure and that the walls, which are typically unreinforced in the Balkan region, are sufficiently strengthened to resist strong seismic ground shaking. In developing the strengthening procedures, the engineer should consider, in case of obvious weaknesses in the structure as a whole, not only to strengthen the damaged portions of the building but also the entire structure. Roof and floor diaphragms may require strengthening and new belts may have to be added. Otherwise, undamaged sections may fail in further earthquakes.

The repair and strengthening solution must utilize materials and techniques suitable to the location of the structure and within the capabilities of the workmen who will perform the work. The solution must also consider all walls and parts of the structure even though they were not damaged in the earthquake.

In conjunction with the development of proposals for strengthening procedures, the following aspects can be considered:

- Reduce the building's weight when possible by removing heavy items such as stone or masonry canopies, balconies, parapets, etc., provided that aesthetics are acceptable.
- Reduce the eccentricity between the center of mass and center of rigidity by adding new walls or closing openings in existing walls.
- Where discontinuities of plan or stiffness occur, provide adequate ties to allow the structure to respond as a single structure or provide a permanent expansion joint with suitable walls for stiffness on each side of the expansion joint, as illustrated in Figure 8.1.
Masonry walls may commonly show cracks due to excessive shear and tensile forces both in the wall proper and near the wall intersections. Considering cracks in the wall proper, typical crack-patterns observed are:

- Diagonal cracks, partially or completely through the masonry width, due to diagonal tensile stresses (Figure 8.2.a).

- Diagonal cracks in masonry piers between window openings, due to diagonal tensile stresses (Figure 8.2.b).

- Horizontal cracks in masonry piers between window openings, due to alternating bending moments (Figure 8.2.c).

- Diagonal cracks above the wall openings, due to shearing and arch-type, load-carrying mechanism together with possible cracks in reinforced concrete lintels (Figure 8.2.d).

Considering the crack pattern near wall intersections, cracks are typically vertical and result of an insufficient interlocking between connecting walls. Such cracks may occur in an interior wall near the junction with an often heavier exterior wall, or in the corner connection of two exterior walls. The repair of wall intersections is discussed in Section 8.3.

Depending on the size of cracks, different repair methods can be used, namely, injection repair, limited or extensive removal and replacement of bricks and stones along the length of the crack, or the replacement of entire wall sections.
Injection Repair of Cracks

Cracks with widths exceeding 0.3 mm but less than 3.0 mm, should be repaired by injection using fluid cement mortar. In special cases, epoxy materials can be considered. Irrespective of the eventual use of strengthening procedures, other than wall jacketing, cracks should generally be repaired.

The sequence of operations in crack injection repair should be as follows:

- Remove the finish coatings, including plaster, from cracked zones and then remove loose mortar or material, dust or other impurities from cracks by air or water jet.

- Bore holes along the crack paths, at 30 to 60 cm intervals, depending on the crack width.

- Introduce sleeves or nipples about 5 cm deep into the holes and fasten them with cement mortar (Figure 8.3).

- Seal cracks with cement mortar, along their whole length.

- Close the sleeves or nipples with stoppers. By removing sleeve stoppers in pairs, the cracks should be cleaned again by air jet and the continuity of injection route checked by introduction of water.

- Inject cement milk or fluid mortar (with a pressure of 3 Mpa). Manual pumps may be used for small quantities, otherwise mechanical equipment is needed. Successive sleeve stoppers are removed in pairs and the fluid is injected through the lower sleeve until it overflows from the upper one. The operation is continued until the filling of cracks is complete.
- Remove sleeve and reapply finish material to conceal crack repairs.

In relatively thin masonry walls in smaller structures where stresses are low, crack repair techniques may be simpler involving the insertion of cement grout or mortar into the cracks by troweling or other appropriate techniques.

**Repair of Large Cracks**

In case cracks are larger than 3 mm in width, cement grout injection can also be used. However, for cracks larger than 10 mm in width or when loose stones or bricks are present adjacent to the crack, repair methods more extensive than injection must be used. The repair procedure selected may naturally involve strengthening the wall. Several methods of repair are typically available.

Cracks which are approximately vertical can be repaired by removing loose stones or bricks adjacent to the crack and adding stitching dogs or steel bars with concrete or replacement stones or bricks using a rich cement grout as appropriate, filling the void (Figure 8.4). This procedure can be repeated on the opposite side of the wall as necessary.
Bricks or stones can be removed from a zone about 15 to 20 cm along an approximately vertical crack and the wall can be reconstructed using elongated bricks or stones which bridge the crack zone and a rich mortar (Figure 8.5). All loose stones or bricks should be removed. An alternative which is an improvement is to fill the void with concrete, thus creating, in effect, a column in the brick wall. Reinforcement can be added to the column consisting of four $\varnothing$ 14 and properly spaced $\varnothing$ 6 hoops or as appropriate to the concrete cross-section.

Inclined cracks of large widths or a relatively dense arrangement of fine cracks, must first be repaired by the injection method or by adding stitching details as described above. Due to their inclined and dense nature, the cracks will not permit reliable stress transmission if only the injection process is utilized, therefore, a series of tie zones can be added to locally strengthen the wall in the zone of cracking. Bricks or stones are removed from a vertical zone 15 to 20 cm wide and 10 to 15 cm deep and the cavity thus formed is filled with concrete, using external formwork to form "columns" or "vertical ribs". Reinforcement consists of hoops and longitudinal bars (Figure 8.6). When vertical ribs extend along the entire height of one floor and if their incorporation into the wall is difficult or impossible, then it is possible to construct, at properly selected points, pairs of ribs (inside and outside), protruding from the wall face (Figure 8.6). Appropriate connection is necessary and these pairs of ribs can be considerably stronger if steel ties pass through the wall interconnecting the two vertical ribs. In relatively slender walls, these tie zones can extend through the entire wall and form strengthening columns or vertical beams to brace the wall (Figure 8.7).
Fig. 8.6

Fig. 8.7
Replacement of Extensively Damaged Walls

Extensive damage may occur to masonry walls which require a portion of the wall to be removed and reconstructed. In such cases, it is important to promptly install temporary shoring to support the floors and walls above which are dependent on the heavily damaged wall for vertical support.

When portions of masonry walls have permanent lateral distortion or humping throughout the width of the wall (Figure 8.8), the distressed portion of the wall must be completely removed and reconstructed. If the wall has spread or humped on only one face, complete reconstruction can be avoided if the vertical face is stable enough to be used as formwork after the humped side has been taken down. Headers are placed in the rebuilt wall using concrete or cement grout to completely fill all voids (Figure 8.9).

Fig. 8.8

Fig. 8.9
8.3 Repair and Strengthening of Wall Intersections

Wall intersections are particularly vulnerable to earthquake damage, resulting frequently in large vertical cracks or separations as the walls are insufficiently interconnected and lack adequate strength to allow proper interaction. Various repair methods may be considered. Considering the basic weakness of masonry construction under earthquake conditions, repair procedures are most often combined with a local strengthening of the wall intersection.

In case of relatively small vertical cracks, repair can be achieved by a repair of the cracks using techniques described in the previous section, with or without some stitching procedures across the crack.

Stone stitching or adding stones across the crack is one method which can be used (Figure 8.10). Adjacent bricks or stones are removed as denoted by "1" and "2" and installing a new brick or stone, denoted by "3", common to both walls. This new stitching stone should be embedded in rich cement grout, at about 70 cm spacing. The gap formed between the two walls is then filled with a rich cement grout. A wire trellis is fastened to both the internal and the external surfaces, and they are plastered with cement plaster. In case reinforced concrete strengthening tie beams or belts were present and were damaged, these tie beams need to be repaired. In case such tie beams were nonexistent, the engineer should consider their introduction to strengthen the structure.

In order to tie the separated wall sections together, steel plates (i.e. 40x4 in cross-section) could be used, embedded in rich cement grout in between two brick or stone layers after some bricks or stones have been removed (Figure 8.11). Such plates can be very effective in reinforcing the corner but they cannot bring the walls back to vertical position. The gap is then sealed and the surfaces covered with wire trellis and plaster, etc., as mentioned above. An alternative is drilling horizontal holes in the masonry through the vertical crack and grouting or epoxing steel rods in the holes. In both of these procedures, the remaining crack should be filled with cement mortar. To reduce the separation prior to repair, tie rods, installed on both sides of the walls, can be used (Figure 8.12). Tightening is carried out by use of bolts and wrench, thus providing a controlled restoration of the walls and
metallic plate for bonding up the wall

Fig. 8.11

crack

metallic plate for bonding up the wall

crack

Fig. 8.12
tying of the corner. Tie bars must be oil-base painted to resist corrosion. If aesthetically permissible, the tie rods can remain in place and the typical crack repair would complete this strengthening procedure.

Particularly in case of stone masonry, the total collapse of a corner region is not uncommon. In such an instance, repair of strengthening requires the introduction of temporary support for the roof or structure above, removal of additional masonry around the damaged area, preparation and cleaning of the contact surface and careful rebuilding (Figure 8.13). Attention should be paid to properly bond the rebuilt part of the wall onto the contact surface. Damaged strengthening belts should be repaired or, if non-existent, should be added. A reinforced concrete corner column properly tied into the intersecting walls could be added to strengthen the wall intersection. Such columns should have minimum reinforcement of 4 Ø 16 and stirrups of Ø 6 at 20 cm (Figure 8.14). In case the introduction of a reinforced concrete column should not be allowed for aesthetic reasons, the use of horizontal reinforcing bars, well extended into the intersecting walls, can be considered to strengthening the corner.

![Diagram of corner with labeled steps](image)

- a - removal or support of part or the roof
- b - additional wall removal
- c - contact surface preparation and careful rebuilding
- d - additional construction of a strengthening belt, thickness 15 to 20 cm, reinforcement 4Ø16 and stirrups Ø6/20

Fig. 8.13
8.4 Strengthening of Walls

8.4.1 Strengthening of Brick Masonry Walls by Confinement

Many structures in the Balkan region are constructed of load bearing masonry walls with the walls built of full bricks with a thickness not less than 25 cm (the usual thickness of the masonry is 25, 38, 52 and 64 cm) in a lime or cement and lime mortar. Floor structures are monolithic and horizontal tie beams or belt courses of reinforced concrete occur only over the load bearing walls with thickness equal to the wall thickness and a height of 25-35 cm. The strengthening of this type of structure can be accomplished by construction of vertical reinforced concrete columns or belt courses at the corners, the intersections of the load carrying walls and in the middle of long walls (Figure 8.15). This achieves a wall panel surrounded by reinforced concrete elements both horizontally and vertically (Figure 8.16) which considerably improves the deformational characteristics of the wall panel. A brittle and non-ductile wall becomes more ductile and its load carrying capacity is increased by several times with the added confinement of the reinforced concrete elements. It should be mentioned that both the behavior and treatment of the masonry strengthened in this way differs significantly compared to framed structures with infill walls. Thus, the following should be observed in the design procedure:

- The vertical belt courses (columns) should not be thicker than the wall.
- The vertical columns or belt courses do not take part in the transmission of the vertical loads, and should be designed with reinforcement to resist the tensile forces due to moments and shear forces of the wall panel.
- The reinforcement in the vertical columns or belt courses should be 4 Ø 14 minimum, while significantly larger percentages of reinforcement should be avoided by inserting more columns.
- The vertical columns or belt courses should be located so that the length to height ratio of the framed wall panels are not larger than 2:1 (Figure 8.16).
- The existing horizontal belt courses are usually not strengthened but if
necessary they are repaired (occasional cracks or partial failures above openings like doors and windows or entrance doors).

The construction of the vertical columns or belt courses for strengthening should begin in the lowest story and proceed upward until the work is completed. The construction typically consists of:
In locations where vertical columns or belt courses are to be constructed, the wall bricks are removed one by one so that the contact zone between the wall and the new concrete is cogged (Figure 8.17).

The concrete of the horizontal belt course at the level of the floor structure should be removed and only the reinforcement left.

A part of the wall on the upper floor is also opened so that the reinforcement of the column can continue and be anchored in the wall above.

Special attention should be given to insure that the vertical columns or belt courses are adequately anchored through the basement to the foundation structure including local strengthening of foundations if necessary.

The reinforcement of the vertical belt course is placed and shuttering forms are placed on the free sides of the vertical belt course, only, while the contact area between the wall and the concrete is dusted out and wetted with water.

Pouring of concrete in the vertical belt courses is done from the upper floor structure.

The placement of concrete in the vertical belt courses at the joints with the horizontal beam is done as a last stage of the repair and strengthening of that story. Cracks should be repaired first followed by any required repairs in places where local crushing or failure took place.
If this type of strengthening is introduced in structures having lightweight or wooden floor structures without horizontal reinforced concrete belt courses, it is necessary that the strengthening include the construction of a rigid monolithic slab above the first floor with reinforced concrete horizontal tie beam belt courses over the load carrying walls and installing bracings at the level of the existing wooden floor structures. This procedure is usually applied only in smaller buildings, up to three stories high. Recognizing the difficulty to construct horizontal monolithic reinforced concrete belt courses, construction of steel bracings along the wall is often more feasible, anchoring the continuous steel bracings into steel plates at their ends and at intersecting walls.

This strengthening procedure of walls can be combined in a particular brick masonry structure with other strengthening procedures, such as "jacketing of masonry walls" or adding new shear walls in the case when there is not a sufficient number of walls.

8.4.2 Strengthening Walls by Confinement with Steel Sections

Structural steel sections can also be used to strengthen masonry walls. Steel sections can be quickly installed and are often used when urgency of repair and strengthening is necessary. Depending on conditions, light steel sections may also be more readily available than shotcreting equipment or other suitable alternatives. The steel sections must be attached to tie beams or belts and the horizontal diaphragms at both top and bottom.

The primary purpose of steel section strengthening is to brace or contain the wall with full height steel sections on both sides of the wall which are attached to the roof or floor diaphragms. Bracing of this type strengthens the wall for forces perpendicular to the plane of the wall and connects the wall to the horizontal diaphragms. It also contains or "baskets" the masonry. This system does not strengthen the wall for in-plane forces.
Figure 8.18 shows a typical application of this procedure. Finish coatings or plaster are removed where the steel section is to be installed and holes are drilled through the masonry at frequent intervals such as 50 cm. It may be desirable to predrill the steel section and use it as a template for the holes to be drilled in the masonry. The steel sections are then installed with bolts between the steel sections. The steel sections are connected to the floors or roof at each end with suitable anchors or straps, depending on the nature of that construction. Cracks and other distress in the masonry wall are repaired by other techniques previously discussed. The steel sections are painted or covered with cement plaster to provide corrosion protection.

The steel sections can be detailed to suit various conditions found in the structure. Figure 8.19 shows one solution at a wall intersection. Figure 8.20 illustrates an isolated masonry pier strengthened with steel angles and straps similar to strengthening of a reinforced concrete column. Steel sections can also be added full height on only one side of the masonry wall with bolts through the wall and plate washers or rosettes on the opposite side of the wall to contain the masonry.
8.4.3 Strengthening Walls with Reinforced Concrete Jackets

For masonry walls exhibiting serious damage and/or when there is a desire to strengthen the building's lateral force resisting system, the addition of reinforced concrete jackets is a logical and often used procedure. The concrete is generally added by the shotcrete method although conventional concrete can be placed. It is most desirable to jacket the wall on both sides with interconnected steel ties to "basket" the damaged unreinforced masonry wall within the new reinforced jackets. However, in some cases, one-sided jacketing of a masonry wall provides adequate strengthening with proper provision for bonding the masonry to the new concrete jacket. The jackets are reinforced with reinforcing bars or welded wire fabric as appropriate.

The initial procedures include providing temporary support of the roof and floors above as necessary and removing all plaster finishes. Loose bricks or stones should be removed and all dust removed with air or water jets. The existing wall can be repaired by the other procedures previously discussed. If the wall is to be jacketed on both sides and if the jacketing is to have sufficient strength to resist both gravity and lateral loads, the existing
Fig. 8.21

wall's cracks might be left unrepaired with only the major damage being repaired, depending on the judgment of the designer.

Figure 8.21 shows the most desirable method to jacket a masonry wall. Holes are drilled at about 50 to 60 cm intervals both horizontally and vertically to pass steel ties which will hook around and engage the reinforcing steel in both jackets. The thickness of the concrete or shotcrete can be about 3 to 4 cm for simple brickwork to 8 cm or thicker for heavy masonry. If concrete is poured rather than using shotcrete, the thickness of the jacket should be at least 10 cm thick for proper placement and pouring holes or ports will also be required. Figure 8.22 illustrates various conditions at wall intersections where an appropriate pattern of ties through the wall must be established to suit the conditions of the strengthening design.

Various modifications of the basic jacketing system can be applied in special circumstances. Figure 8.23 illustrates jacketing above a doorway to strengthen a damaged lintel where strengthening of the entire wall is not performed. Figure 8.24 illustrates jacketing of a pier in a masonry wall when the entire wall is not strengthened. Figure 8.25 shows a jacket on the inside face of an exterior wall with a new concrete floor slab being installed. Anchors through the wall or epoxied into the wall at frequent intervals are essential for proper bond of the existing wall to the new jacket. Figure 8.26 shows a connection through an existing floor to jackets above and below similar to the procedure for adding new concrete shear walls in reinforced concrete buildings. Figure 8.27 shows a condition where a corner has been further strengthened with a thicker jacket and additional reinforcing steel. The condition of an added pilaster for local strengthening is also indicated.
1 - wire mesh
2 - cement mortar jacketing
3 - bored hole
4 - clasp-tie

Fig. 8.22
Fig. 8.23

wire mesh Ø4mm/10cm
on both sides

vertical bars Ø10mm/20cm
horizontal bars Ø8mm/20cm

cement mortar jacketing

clam-p ties Ø8mm
stirrups Ø8mm/20cm

cement mortar jacketing

Fig. 8.24
timber floor is removed and replaced with R/C slab

Fig. 8.25

Fig. 8.26

Fig. 8.27
Alternate systems of reinforcing the jackets can also be used. Figure 8.28 illustrates a system whereby bricks or stones are removed at regular intervals and a reinforcement cage is placed in the chase or void created and concrete or shotcrete is then placed within the chase. This system is best utilized where only one-side jacketing is provided and exposed anchors are objectionable or where heavy stone masonry makes drilling impractical.

Another frequently used alternative when only one-sided jacketing is provided is to cut or chip full height chases into the wall and then add reinforcing steel and concrete in the full height chase along with the jacket. If the full height chase is properly keyed or anchored into the floor or roof system at both top and bottom of the wall, it can provide an effective beam to span vertically and brace the wall for seismic forces perpendicular to the wall.

In smaller structures where continuous jackets are not desired, it is possible to add local reinforced concrete jackets only at wall corners, edges of openings and occasionally at midpoints of walls (Figure 8.29). Although such schemes do save materials, their overall costs are sometimes as high as the complete jacketing approach due to the increased labor and finishing costs.

Shotcrete when installed for reinforced concrete jackets should be installed as described in Section 5.5. The masonry surface should be wetted to avoid water absorption from the fresh shotcrete. If the existing masonry might be unfavorably affected by abundant wetting, a priming coat of cement milk or epoxy resins should be installed on the masonry surface as appropriate. The shotcrete jacket should be properly cured and protected to prevent cracking and detachment from the masonry wall.
One caution with this approach is that the added thickness of concrete may add sufficient weight and overturning forces such that the foundation will require strengthening for increased bearing area. If only isolated walls are jacketed, extremely high stress concentrations and overturning forces may result, requiring special design attention. Designers are also cautioned that jacketing greatly increases the stiffness of the masonry walls and care must be taken not to introduce torsional moments into the structure.

8.4.4 Strengthening Rubble Core Stone Walls with Injection

This method of strengthening applies to damaged buildings constructed of rough or semi-cut stone of poor quality laid in lime or mud mortar or of dry masonry without mortar in the joints. These structures usually have wooden floor and roof systems. Considering the large number of different types of these structures, especially in the Mediterranean part of the Balkan region, and their very low resistance to earthquakes, special attention should be given to the strengthening of these structures. Stone masonry, as a material, is brittle, without ductility, and has low resistance to shear forces. Since it is difficult to increase the ductility of these structures by strengthening, it is necessary to significantly increase the strength resistance of the walls. The type of damage or failure typical to this type of structure is the opening of walls from floors due to their improper connection, and damage or partial failure of walls due to the low load resistance capacity as a result of the low quality of the masonry. Buildings in which walls are not connected in horizontal plane with the floor slab through a belt or at least by steel ties, do not behave as a unit during earthquakes and masonry walls tend to fall outward, failing perpendicular to their plane.

Strengthening of rubble core stone masonry structures can be summarized as connecting the walls of the building at the level of the floor and roof by steel ties on both sides of the wall, anchored at the ends of the walls, and by grouting the walls with cement emulsion or other adhesives.

The first element of strengthening rubble core stone buildings is to interconnect the walls under the floor level by steel ties. The connection is typically achieved by inserting ties, 16 mm diameter threaded steel rods, immediately under the floor structure along all the load carrying walls.
Fig. 8.30

(Figure 8.30). On both sides of the wall, notches are made in the plaster with a width of about 4 cm. The ties are placed in the notches and are bolted on the external side of the wall to an anchorage steel plate which is at least 15 mm thick (Figure 8.31). The length of the plate is slightly larger than the thickness of the wall, while the plate width is a minimum of 20 cm. Anchorage plates of larger sizes should be strengthened by welded webs.
The anchor plates are recessed in a notch in the wall on a bearing surface smoothed by cement mortar. Waterproofing of the anchor plate can be achieved also by cement mortar. The reinforcing bars or threaded rods are tightened by nuts. The reinforcement is tightened until the anchor plate is fully fastened and tight. The bolting and extension of connections can be achieved by bolts and nuts or welding. After tightening of the nuts, they are point welded. The quality of the material and workmanship should comply with the welding regulations and the bolt standards.

Figure 8.32 illustrates alternate details of anchorage of wall ties at the corner of a building. These alternate details require removal of a considerable amount of stone masonry which may be difficult in some cases. It should be assured that enough wall remains with sufficient strength to
positively anchor the ties. Figure 8.33 shows tie and anchor details at a change in direction of a wall. Note that the tie extending inward from the exterior wall face must be positively anchored at a wall intersection or another wall on the opposite side of the building. It is necessary to drill or provide holes through the stone masonry walls to pass the interior tie rods in Figures 8.31 through 8.33 which may be difficult depending on the actual type of stone masonry present. After the rod has been installed, the hole should be filled with cement mortar to reinforce the masonry near the anchorage detail.

In addition to adding the ties, the strengthening of the walls is done by a successive grouting of the entire volume of the walls. Special attention should be given to grouting of connections, cracks and parts of walls between the foundations and the parapets. Where portions of the wall are extensively damaged, rebuilding of those portions of the wall should be performed. At locations of large cracks, the existing plaster should be removed and the surface covered with cement mortar to contain the grout which will be injected. Fast drying mortar grouting pipes are then placed along the height of the wall at cracks at a distance of 30 to 60 cm, depending upon the width of the cracks. Strengthening of other wall areas which are not cracked is done by placement of grouting pipes in drilled holes at spacings of 50 to 80 cm following a mesh pattern. Before grouting, it is necessary to fill the porous spots not covered by plaster of the wall surfaces with cement mortar 5 mm thick. Before grouting, cracks and joints are wetted and washed by clean water.

Grouting is accomplished under pressure of 1.5 to 3 atmospheres, while at the final stage when the pipes are filled in, the pressure is retained for five minutes in order to drain the water from the grouting pipe. The grouting starts at the lowest spots of the wall and proceeds upward, after all repairs of walls and placement of horizontal ties have been accomplished. Grouting pipes are removed four hours after the grouting. If grout appears during grouting in the adjacent pipes, they should be closed temporarily.

Special equipment consisting of a grouting pump, hoses and ports is required for accomplishment of this procedure. This equipment represents a set of simple and inexpensive devices which require proper maintenance and operation.
Grouting for injection usually consists of 90% cement, 10% fine grained tuff and sufficient water which will provide a volume ratio of water to dry components of 1.0 at the beginning of grouting and then increased to the ratio of 0.8 as grouting proceeds. This grouting procedure typically requires a quantity of dry adhesive mass of 120-125 kg per m$^3$ of wall, depending upon wall porosity.

When this method of strengthening is used, it is desirable for the floor structure to be constructed as a monolith reinforced concrete slab with all the required belt courses, so that the overall strength of the structure is increased. Wood floor systems can also be strengthened as discussed in Section 8.5. This method of strengthening stone structures can be used in combination with other procedures such as adding shear walls or strengthening shear walls by jacketing of some wall parts within the structure. In this case, when new elements are introduced into the structure, the foundation structure should be checked and strengthened, if necessary.

8.5 Strengthening Horizontal Diaphragms and Tie Beams

Frequently, one of the greatest deficiencies of masonry structures in seismic regions is a lack of proper horizontal diaphragms and interconnections between the diaphragms and the masonry walls. Buildings need to be tied together along wall lines and across the building at all floor and roof levels to properly survive in a seismic environment. Tie beams or belts are commonly provided in the masonry structure along each masonry wall at each floor and roof to affect a more even distribution of lateral force to the masonry wall and tie the structure together so that it performs as a single unit. The tie beams also act as chords for the diaphragm. If existing tie beams or ties between wall and floor or roof diaphragms are damaged in an earthquake or are found to be lacking or weak at a particular location, appropriate strengthening details must be developed to suit the actual conditions. Floor or roof diaphragms may also need strengthening to increase their stiffness and/or strength. This section will discuss the addition of such features when they are not present in the structure.

There are many ways to install a new tie beam or belt in masonry walls at roof or floor level. Figure 8.34 shows a new reinforced concrete tie beam being
added in two stages so the wall supports the roof above. Longitudinal steel in the tie beam or belt must be continuous and properly spliced at any discontinuities in the construction. Transverse walls should have similar tie beams or belts anchored to the exterior tie beam. Figure 8.35 shows temporary support of the roof allowing removal of the top of the masonry wall and construction of a new belt or tie beam. Trusses are anchored to the tie beam to brace the top of the wall. Alternatively, if there is an uninterrupted gap between the wall top and the rafters, the reinforced concrete strengthening belt can be constructed without lifting or supporting of the roof (Figure 8.36). In such a case, the truss ties can work as a tie for forces perpendicular to the wall, after an appropriate anchoring onto the belt. The truss ties must be locally protected against moisture. The part of the timber that is to be covered by concrete can be jacketed with 2 cm expanded polyester which is removed following the setting of concrete to allow ventilation in order to avoid rotting of the timber. However, a steel connection between the truss and the belt or tie beam must be provided to transfer forces which tend to pull the wall outward. A further alternative similar to jacketing is shown in Figure 8.37. This method can be used also at lower floor levels.

Floor and roof construction often consists of wood boards supported by timber beams or trusses. Straight sheathed wood diaphragms of this type are weak for lateral loads and stiffening and strengthening is often necessary to provide an acceptable diaphragm at that level. Specially manufactured sheathing material such as plywood is ideal for this purpose as an overlayment if such material is available. Obviously, the material must be adequately nailed to the existing floor or roof construction. Wood floors can also be considerably stiffened by adding new wooden boards or planks perpendicular to the existing flooring boards (Figure 8.38). Each new plank must be nailed to each board it crosses with enough nails to make the boards work together as a diaphragm, at least two and preferably four nails from each new plank to each old board below. The new planks can also be installed diagonally, which forms a sort of
Fig. 8.36

cover removal

casting of concrete belt

2cm polyester jacket around the timber

R/C strengthening belt

Fig. 8.37

4Ø10/100m

mesh

0.60

duct

4Ø10/100m
horizontal truss work between the straight and diagonal boards and can achieve considerable strength and stiffness. It is also possible to stiffen the wooden floor with a thin concrete layer above the existing wood construction (Figure 8.39). The concrete is typically 4 cm thick and reinforced with a steel mesh Ø4/15x15. Nails driven, at a slant are used to anchor the new concrete layer to the existing wood and steel anchors while steel anchors are grouted in drilled holes at the masonry walls.
Possibly, the most important feature of strengthening a masonry structure with wood floor construction is providing adequate ties between the exterior walls and the diaphragm to prevent the wall from falling outward. There are various methods to install or strengthen ties between masonry walls and floor or roof diaphragms. Figure 8.40 shows steel ties which extend across the building and are anchored with plates on the exterior face of the masonry walls. Steel ties similar to those discussed in Section 8.4.4 can also be used. Figure 8.41 shows a steel strap detail connecting an existing masonry wall to a horizontal wood floor beam. This detail is convenient when the floor beams frame perpendicular to the exterior wall and can be strengthened by the use of bolts rather than nails to the floor beam. Figure 8.42 shows a similar detail when the floor beams are framed parallel to the exterior wall. In this detail, the straps must extend along the floor far enough to provide sufficient attachment to the floor planking and a wood or concrete fill may be necessary to provide a smooth finished floor. The wood diaphragm must have sufficient tensile capacity beyond the straps for these details to be effective and prevent the diaphragm from pulling apart at the first interior wall or support.

![Diagram showing connections between masonry walls and floor or roof diaphragms](image-url)
8.6 Strengthening of Foundations

If masonry walls have been damaged due to differential settlement or sliding caused by the earthquake, action should be taken to remove the cause for such damage. Foundation strengthening can also be required to increase the bearing area when the strengthening or reconstruction increased the building's weight and/or increased overturning forces. It may also be desired to provide structural ties at the foundation to assist the building behave as a single unit when the ground movement occurs.

Stage-by-stage underpinning is possible by means of placing concrete blocks (length $b = 0.50$ to $1.00$ m, spacing $\geq 2b$) in order to avoid occurrence of differential settlement (Figure 8.43). Potential sliding of a foundation can be reduced by constructing reinforced concrete "bridles", especially in

---

**Construction of underpinning blocks in two stages**

**Section A - A**

**Fig. 8.43**

**RC "briddle"**

**Fig. 8.44**
sloping ground (Figure 8.44). These "briddles" are constructed deep into the soil, to the downslope side of the foundation, in contact with it at its lowest point, and running parallel to ground contours. Another method is to build a reinforced concrete belt all around the building at the foundation level (Figure 8.45) or to build a tie beam along the inner side of the foundation (Figure 8.46). These last three schemes can also provide a positive foundation tie at the building's perimeter with continuous longitudinal reinforcing bars provided.

**Fig. 8.45**

bricks or stones are removed every 2.50 m to anchor the belt in concrete-filled chases

**Fig. 8.46**
9. VERIFICATION OF DESIGN AND CONSTRUCTION

9.1 General

The verification of the design and the construction is an essential part of any seismic repair and/or strengthening project. Verification of design is the thorough review by the designer and an independent review that the design criteria and solution are appropriate and that the structural calculations and details have been properly prepared. Verification of construction is the inspection and testing of materials and procedures of construction and the in-depth review of the details of construction to assure that the design has been properly implemented.

While the verification of design and construction is important for all construction projects where a failure would affect the health and safety of the occupants, it is particularly important in seismic repair and/or strengthening projects. The design criteria and solution for such projects requires considerable engineering judgment and careful attention to details which are quite different from those commonly encountered in the design of new structures. Detailed inspection and verification of the construction procedure is also particularly important since the Contractor will be utilizing some construction materials and techniques not normally encountered in new construction and unexpected difficulties and questions will be encountered in the construction process requiring design decisions.

9.2 Design Verification

Every design organization or company should have its system for the review of each design. Seismic repair and strengthening projects require full utilization of such review systems if they are to result in successful projects.

For major and important structures, the advice of other experienced engineers is helpful in reaching decisions regarding repair and strengthening projects. Once the data regarding the original construction and condition of the structure have been documented and the designer has developed alternatives for the repair and strengthening solutions, a review committee should be assembled to contribute their ideas and judgment regarding the proposed criteria and solution for the project. This input should be utilized by the designer in preparing the final design.

When the design has been completed and the designer has thoroughly reviewed his work for completeness, a check of the work should be performed by another engineer. This check should verify the criteria, check the accuracy of the calculations and verify that the design drawings correspond to the calculations. The completeness of the drawings and appropriateness of all details should also be checked.

It is recommended that an independent review of the design be conducted by an agency not under the control of the design organization. In many countries this independent review is a government function which verifies the criteria and checks the calculations and drawings for conformance with the criteria and the building codes and regulations. Independent consultants might be hired to
perform this review. If the government does not require such an independent review of the design, the owner may wish to hire an independent design consultant to perform this review. Obviously, any comments or questions raised in the review must be adequately answered by the designer, with appropriate changes to the drawings as needed.

Independent checks and reviews of the design must not be viewed as an expression of doubt regarding the designer's competence. Designers should welcome an independent review which might catch errors or point out items overlooked in the design process.

9.3 Construction Inspection

The construction inspection is conducted by an individual, agency or firm in accordance with conventional practice. An experienced construction inspector knowledgeable of the materials and techniques involved must be utilized on repair and strengthening projects. The design engineer should be involved in the inspection process and answer all questions affecting the design which arise during the construction. This is extremely important for such projects as many unexpected conditions will be encountered during the construction process, and only the design engineer or his representative can properly assess the significance of these unexpected conditions and make the proper decision regarding necessary changes to the design.

A structure which has been damaged in an earthquake will have hidden damage which will be discovered only after finishes have been removed and the construction process is well underway. Unexpected conditions will be uncovered during construction because it is impossible to anticipate as-built conditions exactly. Piping or utility lines will interfere with a strengthening scheme or existing reinforcing will not conform to expected conditions, requiring changes to the design details during construction. Because of his familiarity with the design and its elements, it is essential that the design engineer, make the necessary changes to suit the actual field conditions.

9.4 Construction Verification

The verification of construction for seismic repair and strengthening projects is similar to the inspection process for new structures. The quality of materials is verified by sampling and testing the materials supplied while an inspector verifies that the materials are properly installed in the correct location. The differences from inspecting new construction involve the testing of special materials such as epoxy and shotcrete and the verification of existing conditions and the suitability of the details to the conditions encountered. The inspector should ascertain that the workmen are qualified to perform specialized tasks involved in repair and strengthening projects.

Traditional construction materials such as concrete, mortar or grout are sampled and tested as in new construction, with strength being verified at a specified age by compression tests of cylinders or cubes of the material. Steel strength and properties are verified by mill reports and testing of samples. Welding is inspected as in new construction with certified welders performing only those welding procedures for which they have been certified.

Shotcreting should be continuously inspected by a qualified inspector who should check materials, forms, reinforcing, gunning equipment, application of material, curing and protection against freezing. Quality control of shotcrete is more difficult than for conventional concrete since it is affected not only by the accuracy of batching but, more importantly, by the skill and continued
care of the crew applying it. Each layer of shotcrete should be systematically sounded with a hammer to check for voids. It is generally not feasible or desirable to core the structure to obtain specimens for regular control tests. Therefore, small unreinforced test panels, at least 38 cm square and 8 cm thick, should be periodically gunned, and cores or cubes extracted for compressive tests and visual examination. In addition, several cores may be taken from the structure if desired to verify the quality of material in place. Such cores should be taken as early in the job as practicable to detect deficient workmanship.

Epoxies and other resins are usually tested by the manufacturer who verifies their strength and properties with appropriate certification. The inspector should observe the mixing and handling procedures and verify that all work is in accordance with the manufacturer's recommendations and the project specifications. Besides verifying that the resin is properly mixed and used within its pot life, the other important aspect of epoxy injection is verifying that the material is effectively injected to fill the cracks. The only practical procedure to verify the injection is to core drill across the crack and inspect the core to determine if the crack was properly filled with epoxy. If desired, the core can be tested and the failure should not occur along the epoxied plane of the crack.
10. CASE STUDIES
10.1 BULGARIAN EXAMPLE OF REPAIR AND STRENGTHENING OF A SLIP-FORMED RESIDENTIAL BUILDING*

10.1.1 Introduction

Residential buildings with various structural systems were damaged as a result of the Vrancha earthquake on March 4, 1977 in the northern part of Bulgaria. The intensity in the affected zones according to the MSK scale was from VI to VIII. The repair and strengthening of a damaged slip-formed residential building in Sofia is presented. The structure was strengthened for a seismic MSK intensity of VIII. The building was founded on sandy clay with a design bearing stress of 0.25 MPa.

10.1.2 Description of the Building

The 9-story structure was built using slip-formed construction. The damaged building is a reinforced concrete shear-wall type structure of rectangular plan with dimensions 14.00m x 58.40m. The location of the existing shear walls as shown in Fig. 10.1.1 indicates an eccentricity between the centre of stiffness of the longitudinal walls and the centre of mass.

According to the construction process the walls are built first using slip forms. The slabs are then placed and connected with the walls by specially detailed reinforced concrete dowels. The slab can be assumed to act as a rigid diaphragm transferring the seismic loads to the shear walls.

For the existing structure concrete C 20 with cubic strength of 20 MPa was used. Steel A-I with design strength of 210 MPa was applied for the slab and web reinforcement, while steel A-II with design strength of 270 MPa was used for main wall reinforcement.

10.1.3 Damage of the Building

As a result of the earthquake on March 4, 1977 the columns located along line C (Fig. 10.1.1) suffered extensive damages in the lower stories. Local crushing of the concrete and buckling of the reinforcement was noted. In the higher stories these columns have been damaged only slightly. Slight cracks were observed in the wall plaster. A thorough inspection of the structure indicated that the damages were mainly due to poor execution of the construction resulting in insufficiently compacted concrete, initial cracks, etc. Torsional motions resulting from an eccentricity between the mass and stiffness centres noted above may also have contributed to the observed damages. However, it is reasonable to assume that the transversal shear walls were capable to resist these torsional moments well.

* The design of the strengthening procedure described, was performed at the Design Office "Sofaproject", Sofia.
10.1.4 Design Analysis

Considering the geometrical characteristics of the structural elements and the material properties, the bearing capacity of the damaged structure was analyzed. As a result, sufficient strength was verified for the vertical loads. However, for lateral seismic loads the design was found inadequate.

On the basis of the analysis, it was decided to construct five additional frames against the face of the building along line C (Fig. 10.1.1). The frame location on the outside of the structure eliminated the necessity of evacuation of the inhabitants. The sizes of the newly added frame members were chosen as large as possible in order to achieve a frame stiffness compatible with the stiffness of the shear walls of the existing building and to reduce or eliminate the eccentricity between the building mass and stiffness centres. The scheme of the newly added frames and the geometrical dimensions of the strengthened members are shown in Fig. 10.1.2.

The strengthened building structure is designed in accordance with the Bulgarian Code for Building in Earthquake Regions and the Bulgarian Concrete and Reinforced Concrete Structure Design Code. According to the Code provisions, the internal forces in the newly formed reinforced concrete frames are calculated for the combined action of the existing and the newly added structural elements. The seismic forces are distributed among the shear walls and newly added frames assuming rigid floor diaphragms. The frames along line C carry about 30% of the total seismic load in longitudinal direction. The connections between the existing structure and the newly added elements are proportioned to transmit both live and seismic loads.

10.1.5 Repair and Strengthening Procedures

According to the damage patterns and the results of the analysis of the building structure, the damaged columns were repaired before building the newly added frames. Local damaged concrete in the existing columns was taken away. Damaged reinforcement was replaced by new reinforcement, which was welded to the existing bars (Fig. 10.1.3).

The connection between the existing and the newly added elements is realized by: roughening the existing concrete surfaces; attaching the new reinforcement to the existing one by reinforcement ties and anchor bolts where it is necessary. In this way, the interaction between the existing and new structures is realized.

The construction followed step by step starting from the first story. The strengthened beam and column sections of the frames are shown in Figs. 10.1.4 and 10.1.5 respectively. The frames are supported on the existing mat footing as shown in Fig. 10.1.6.

The materials used for the strengthening are the following: concrete C 30 with cubic strength equal to 30 MPa and steel for web reinforcement type A-I and for longitudinal bars A-III with design strength of 210 MPa and 360 MPa respectively.

The approximate cost of the strengthening of the structure is about 4% of the building replacement cost.
REFERENCES

Frame type F-2
number 3

Frame type F-1
number 2

Fig. 10.1.2
Fig. 10.1.3

Fig. 10.1.4
Fig. 10.1.5

1. existing reinforcement
2. new reinforcement
3. stirrups
4. existing concrete
5. new concrete
6. anchor bolt

Fig. 10.1.6

1. existing structure
2. added structure
3. anchor bolt
4. epoxied anchor bolt
GREEK EXAMPLE OF REPAIR AND STRENGTHENING OF AN APARTMENT HOUSE
IN ATHENS AFTER THE EARTHQUAKES OF 1981*

10.2.1 Abstract

The earthquake of 24-2-1981 caused quite a few damages in several buildings in the area of Athens. It is significant that in Athens we did not have any total collapses and the damages were comparatively small.

This example describes the damages, the repair and the strengthening of a typical apartment building in Athens. (In the Halandri region).

10.2.2 Description of the Building

The building (600 m²) consists of a ground floor and 4 stories with a basement under the main entrance. The building has a reinforced concrete frame and footings for foundation.

10.2.3 Description of Damages

The earthquake caused damages to the bearing structure, the lightweight partitions and the external walls of the building. The position and magnitude of the damages in the ground floor (Pilotis) are shown in Figures 1, 2 and 3.

10.2.4 Research and Checking

In order to take the necessary decisions and to have data for the calculations for the repair and strengthening of the bearing structure of the building, the following research and calculations were made:

i Geotechnical study
ii Detailed checking of the reinforced concrete structure
iii Checking the strength of the concrete by cores taken from some columns of the building.
iv Calculations of the seismic behavior of the building, as constructed.

10.2.5 Necessary Repairs and Strengthenings (Figures 4 through 10)

i Before commencing any research work and checking, the ground floor of the building was supported, as shown in the photographs. In this way, any further damages from possible continuation of the seismic activity were prevented.

ii Repair of all the cracks and disorganized sections of the concrete structure took place by means of epoxy resin injections, resign grouting, and polymer concrete, according to the magnitude, the width and the place of the cracks.

* This design was done by OMETE Ltd C. Kostikas, G. Iakovides, P. Kremezis.
iii Construction of jackets 10 cm thick with shotcrete in all the damaged columns.

iv Construction of new shear wall in order to increase the rigidity of the building.

v Strengthening of the shear walls around the elevator on all levels.

vi For the strengthening of the foundation, the construction of grade beams on the level of the necks of the footings around the whole perimeter of the building was decided.

10.2.6 Structural Analysis

(Calculation of stresses and forces transfer mechanism)

Since the unloading of the dead weight from the elements of the existing structure was impossible, the bearing structure was calculated twice for the vertical forces (the existing bearing structure with the dead load and the redesigned one with the live load).

For stresses due to seismic horizontal forces the analysis of the structure was done for all the loads on the redesigned structure, in two main directions, and considering the building as a multistory frame.

For the calculations, the following assumptions were made:

i In the calculation of rigidity, columns with jackets were considered as single sections.

ii The connection of shear walls with adjacent columns is secured with bent down bars that can transfer the forces, and the section (shear wall and adjacent columns) is considered for its rigidity as single.

iii The transmission of the horizontal forces from the disc of the slab to the new shear walls is secured by means of the bent down bars*

* The calculation of the bearing capacity of the bent down bars is done according to B.H. Rasmussen—(F Leonhard, Bank 2 Special Chapter of Reinforced Concrete).
Example

Calculation of Strengthening Shear Walls

1. Shear wall T5-T6

\[
P_5 = 139.4 T
\]

\[
K_5 \frac{30 \times 100}{6 \phi 16 + 2 \phi 8/120}
\]

\[
\alpha \theta
\]

ELEVATION

\[
B_0 \phi 26/16
\]

\[
B_0 \phi 18/15
\]

FOUNDATION

\[
180
\]

\[
300
\]

\[
345
\]

\[
180
\]
For the structural analysis shear walls T5 and T6 were considered as one with dimensions 570 cm x 30 cm.

The following forces were computed:

\[ M = 462 \text{ Tm} \]
\[ P = 230 \text{ T} \]
\[ Q = 115.5 \text{ T} \]

1. **Bending (Reinforcement)**

- The necessary reinforcement is calculated for bending with compression (middle eccentricity);
- The existing reinforcement in the columns is taken into account;
- In cases that no extra reinforcement is needed, the jackets of the columns are reinforced with \( \phi 14/15 \) with ties \( \phi 10/15 \).

2. The shear walls are reinforced with two meshes.

The same reinforcement is placed horizontally and vertically. The percentage of applied reinforcement is the maximum needed in one of the directions.

**Shear stresses on the wall due to \( M \).**

\[ V = \frac{M}{L} = \frac{462}{5.70} = 81.05 \text{ T} \]

\[ \tau_1 = \frac{V}{d x h} = \frac{81050}{30 \times 350} = 7.72 \text{ kp/cm}^2 \]

**Shear stresses due to \( Q \)**

\[ \tau_2 = \frac{Q}{0.85 \times d x D} = \frac{115500}{0.85 \times 30 \times 350} = 7.95 \text{ kp/cm}^2 \]

\[ \mu_{\text{min}} = \text{max} (\mu_v, \mu_h) = \frac{\tau - 0.4 \tau_{\text{e}}}{\sigma_{\text{e}}} = \frac{7.95 - 0.4 \times 7.0}{2.4} = 2.15\% \]

Minimum percentage of shear reinforcement \( \mu = 3.125\% \)

\[ f_{\text{e min}} = \frac{3.125 \times 30 \times 100}{1000 \times 2} = 4.69 \text{ cm}^2 / \text{M} \]

3. **Wall and Beam Connection**

The connection is done by means of bent down bars. They have to bear the shear forces. According to Prof. Leonhard and on tests by Rasmussen the bearing capacity of a bent down bar that is embedded in concrete is calculated by:

\[ P = 2.5 \phi^2 \sqrt{B_p \times B_s} \]

\( \phi \) = diameter of bent down bars
\( B_p \) = yielding point of steel (kp/cm²)
\( B_s \) = strength of reinforced concrete (cube in 28 days) (kp/cm²)
\( P_u \) = ultimate load

Embedded length of bent down bars 1.6\%)

A safety factor of \( V = 5 \) is suggested by Rasmussen that assures that the deflection of the bent down bar on its loading point and for the applied force will be \( \leq 0.005 \phi \)

\[ P_{\text{e}} = \frac{1}{5} P_u \]

so for \( B_p \sim 0.8 B_{\text{min}} \) and \( B_s = 4200 \text{ kp/cm}^2 \), we have for \( \phi 26 \)

\[ P_{\text{e}} = 3.08 \text{ MP} \]
Because the load is considered exceptional, we cannot have an increase of permissible force by 20%.

\[ S_o P'_{ex} = 1.2 \times 308 = 370 \text{ MP} \]

And the total number of the bent down bars

\[ \eta = \frac{Q}{P_{ex}} = \frac{115.5}{3.7} = 31 \]

31026/16 are applied.

Shear Wall T19-20

\[ P_{g} = 25.4^T \quad P_{20} = 47.5 \]

The calculation of bending, shear and bent down bars is done in the same way as for T5-T6.

Bent Down Bars of Columns

The connection of the shear walls and the columns is done through bent down bars.

\[ V = \frac{M}{L} = \frac{166.6}{2.95} = 56.5 \text{ TM} \]

\( \phi 20 \) are used with a working capacity \( P_{ex} \leq 1.80 \text{ MP} \)

and because the earthquake is considered an exceptional load

\[ P'_{ex} \leq 1.2 \times 1.80 = 2.20 \text{ MP} \]

\[ \eta = \frac{56.5}{2.2} = 26 \]

26020/13 are applied.
FIG. 2 COLUMN DAMAGES - COLUMNS K1 AND K13
FIG. 3 COLUMN DAMAGES - COLUMN K7
FIG. 4.
STRENGTHENING
JACKETS
NEW SHEAR WALLS
FIG. 5. TYPICAL DETAIL SHOWING A SHEAR WALL ADDITION BETWEEN COLUMNS WITH JACKETING
TYPICAL DETAIL OF TOP CONNECTION BETWEEN SHEAR WALL AND BEAM

TYPICAL DETAIL OF COLUMN JACKETING WITH GUNITE

FIG. 7 TYPICAL DETAILS
FIG. 8 CONSTRUCTION OF NEW SHEAR WALLS K19 AND T19-20
FIG. 10. SECTIONS THROUGH FOUNDATIONS
10.3 YUGOSLAV EXAMPLE OF REPAIR AND STRENGTHENING OF "MAKSIM GORKI"
ELEMENTARY SCHOOL BUILDING IN TITOGRAD*

10.3.1 Abstract

This design example of repair and strengthening was presented to the Seminar of Working Groups D and E of the UNDP/UNIDO Project RER/79/015 "Building Construction under Seismic Conditions in the Balkan Region", which was held in Titograd in November 1982.

The author presented at the Seminar the approach developed by the Republic Institute for Town Planning and Design, Titograd, for the design of repair of public buildings (schools) with an orthogonally framed structural system. The design of the repairs, which were carried out in 1981, was presented at the seminar. This presentation offers a practical design example and indicates the considerations and decisions made in repairing and strengthening this building.

10.3.2 Introduction

The Montenegro earthquake of April 15, 1979 and the series of aftershocks caused considerable damage to both structural elements and nonstructural infill of the "Maksim Gorki" school building in Titograd, constructed in 1962.

The design according to which the structure was constructed in 1962 was available. This information together with studies of the foundation soil properties and the quality of the structural elements, served as the basis for developing the proposed design for the repair and strengthening of the structure.

Papers which were prepared during construction at the site had not been preserved. Therefore, it was not possible to determine completely the difference between the design and the actual construction.

In order to define acceptable criteria for the seismic risk level and the required resistance to future expected earthquakes, a study of the seismic parameters through geotechnical soil investigations was carried out.

The quality control of the concrete was carried out by taking concrete cylinder samples of the existing structure for all vital structural elements.

Studies and investigations on the soil conditions, the quality of construction materials and workmanship were carried out by specialized and authorized organizations and institutions.

* Example prepared by Vladimir Stankovic, Principle Design Engineer, and Radovan Radovic, Republic Institute for Town Planning and Design, Titograd, Yugoslavia.
10.3.3 Description of Structure and Damage

The existing structure was designed and constructed as a framed structure with hollow brick infill walls, constructed after completion of the concrete frames. The frames were oriented in both orthogonal directions, (x and y) with beams more loaded in the transverse (y) direction (Fig. 1).

The floor structure was constructed of prefabricated beams with a non-reinforced concrete slab. In the larger-span portions the floor is a monolithic web floor with a reinforced concrete slab (webs placed at distances of 0.5 m).

The foundation structure consists of continuous and isolated footings at a depth of about 1.40 m below ground level.

The expansion joints, placed at the high classroom section part and gymnasium, have a thermal function but do not meet seismic criteria.

Tests of neither the complete structure nor the structural elements had been carried out. Also, data from the built-in material quality tests carried out during construction were lost. According to the reports of persons involved in the construction of the building, the quality of the construction complied with the code provisions effective at that time.

The commission for damage classification, which inspected the building after the April 15, 1979 earthquake, marked it with one orange line, which meant the structure could not be used temporarily and had suffered slight structural damage.

The design of the structure and the structural elements had been based on the usual static analysis method, effective in Yugoslavia in 1962. Accordingly, the structure was designed for S = 0.036 g.

Based on previous investigations and according to data published by the Seismological Observatory in Titograd, it can be concluded that during the April 15, 1979 earthquake, the structure sustained the effects of a VII MCS degree earthquake, except for the columns at the gymnasium.

In the analysis, necessary to assess the capacity of the basic structure and to develop the repair design, the global value of the seismic coefficient was taken to be $K = 0.192$; resulting in a total horizontal seismic force $S = 0.192 \, G$ with $G =$ weight of the structure. The results thus obtained were unsatisfactory, especially the displacement which, for the stated intensity level earthquake, would reach a magnitude of 56 mm in frame $R_{2y}$.

The structural requirements for repair and strengthening necessitated changes in the building functions in order to justify the repair expenditures.

10.3.4 Method of Repair and Strengthening and Analytical Approach

On the basis of the principles of the Code and Regulations for Repair of Structures Damaged by the Montenegro Earthquake, static and dynamic analyses of the structure were carried out considering the site soil conditions and site seismic characteristics.
According to the accepted principles for economically justified repair, and based on the observed damage mechanisms caused to structural elements, the proposed strengthening of the structure, using an acceptable factor of safety should ensure the following seismic risk level:

a) complete absence of nonstructural damage under slight earthquakes, which can frequently occur during the serviceability period of the structure.

b) no structural damage and minimal damage to non-structural elements (infill and partition walls), under moderate intensity earthquakes, which would occur, with a return period of 50 years, several times during the serviceability period of the school building.

c) the structure should sustain the strongest expected earthquakes (return period of 200 years) with partial and confined damage to vital structural elements and considerable damage to non-structural elements.

It is obvious from the observed damage degree and from analysis of the damaged structure that the building, as constructed in 1963, did not meet the criteria under (a) and only partially those under (b). Therefore, it was necessary to strengthen the existing structural system, especially the vital structural elements.

The method of repair and strengthening is imposed by the need to decrease the deformations under the earthquake effects and to prevent frame columns from taking seismic loads, particularly bending moments. To meet these objectives, reinforced concrete shear walls, \( d = 0.10 \) m and \( d = 0.15 \) m have been added to the existing structural system (Figs 1 through 5). The number and the location of the shear walls was determined in compliance with the function of the building (school).

For determination of the expected seismic effect on the repaired and strengthened structure, dynamic analyses under actual earthquake effects, obtained by site seismicity investigations, have been carried out.

The basic data obtained from the site investigation indicated an expected maximum ground acceleration of 26% g. For the time history analysis, the record of the Parkfield 1966, N-S component, with a maximum ground acceleration of 0.5 g, has been taken. Consequently, an intensity of 50% of the Parkfield earthquake was used in the structural analysis.

The vibration periods of the repaired and strengthened structure have been obtained as follows:

For y direction:

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<tr>
<td>II</td>
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<tr>
<td>III</td>
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For x direction:

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<tr>
<td>III</td>
<td>0.041</td>
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</table>

Further dynamic analyses, using the Montenegro earthquake records taken at the "Oliva" hotel in Petrovac, N-S component, and at Ulcinj, E-W component, produced basic structural response data, as presented in Table 1.
Table 1

<table>
<thead>
<tr>
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<th>Storey height</th>
<th>Period T, sec</th>
<th>Structural response</th>
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<td>Petrovac Earthq. N-S</td>
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<td>Max. displacement</td>
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<td></td>
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<td></td>
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<td>Y-Y</td>
<td>3</td>
<td>0.1</td>
<td>0.189</td>
<td>3</td>
</tr>
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<td></td>
<td></td>
<td>0.2</td>
<td>0.362</td>
<td>3</td>
</tr>
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<td>4.67</td>
<td>0.3</td>
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<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.4</td>
<td>0.966</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>3.67</td>
<td>0.5</td>
<td>1.377</td>
<td>3</td>
</tr>
</tbody>
</table>

The results from the analysis indicated that the displacements of the repaired and strengthened structure, under peak ground acceleration of 0.5 g, are within the permissible range and that damages to the structure and infill walls under a moderate intensity earthquake will not occur.

10.3.5 Sequence of Construction and Safety Measures, Detailing and Cost Analysis

Because of possible problems associated with the safe execution of the repair and strengthening works and the possible discovery of damages other than those observed originally and considered in developing the repair design, the repair and strengthening of damaged structures is a complex engineering process.

Considering, in this case, that the structure has several expansion joints, it was decided to carry out independently the works in the classroom block and ground floor annex and in the gymnasium section.

For a safe execution of the repair, it was considered necessary that all users would vacate the building and that all valuable movable inventory and equipment be removed for later use.

Also, in the preparatory stage, it was deemed necessary that all the following safety measures be taken, namely:
- turn off all electric switches;
- close all water supply taps;
- open all doors for faster evacuation in case of emergency;
- train persons to be able to use the main and emergency exits in case of emergency;
- waste material is not allowed to be kept in the building and should be removed;
- persons who are not directly involved with repair works are not allowed on the site; and
- establish a portable first aid kit with basic medical supplies.

The execution sequence of the works, ensuring a maximum safety for both the building and the workers, was as follows:

- construction of expansion joints at the annexes,
- construction of foundation for transverse shear panels in y-direction,
- construction of foundation for shear panels in x-direction,
- construction of transverse diaphragms to the roof level,
- construction of longitudinal diaphragms to the roof level and
- construction of remaining diaphragms.

It should be noted that during the repair and strengthening period the building sustained several earthquakes of slight intensity.

The structural strengthening through the proposed method requires full integration of the existing and added elements. Figs 6, 7 and 8 show details of connections between existing and added reinforced concrete wall elements.

By adding the rigid reinforced concrete walls, a redistribution of the horizontal force is provided with the walls resisting the larger part of the horizontal seismic forces. Hence, a considerable overturning moment at the foundation will result, requiring a strengthening of the existing foundation and expanding the same (Fig. 9).

The distribution of the repair costs, as per January 1, 1981, was as follows:

1. Design and field research 4.10
2. Investigations and analysis 2.05
3. Taxes and supervision 4.10
4. Tearing down and opening of walls 4.28
5. Earth works 2.05
6. Masonry works 5.15
7. Concreting of foundation 13.14
8. Concreting of shear panels 4.51
9. Roofing works 11.99
10. Finishing of floors and carpentry (glassware) 6.46
11. Painting 4.26
12. Water supply and sewage piping 1.07

(carried to next page)
The total costs of repair works were 30% of the value of the building. It should be noted that the central heating system is new and should actually be excluded from the total repair costs.

10.3.6 Conclusions

The presented design of repair and strengthening of the building points to a rational way of strengthening a reinforced concrete framed structure.

By adding reinforced concrete walls, the characteristics of the whole system changes, i.e.,

- it becomes a rigid system with lower natural periods and reduced displacements;
- the existing frames do not suffer bending moments under earthquake conditions;
- improved stability of structure and remaining non-structural infill elements.

Further analyses have shown that displacements are proportional to damage and vulnerability.
Fig. 1. Plan of ground floor with distribution of existing frames and strengthening walls

Fig. 2. Elevation of middle longitudinal frames with shear walls R-3-x
Fig. 3. Elevation of external cross frame with shear wall R-11-y

Fig. 4. Elevation of middle cross frame with shear wall R-8-y
Fig. 5. Elevation of middle existing frame without strengthening
Detail of joint of column and diaphragm at corner

Fig. 6. Detail of connection of column and shear walls at the corner
Detail of joint of beam and diaphragm

Fig. 7. Detail of connection of beam and shear wall

Detail of joint of column and diaphragm

Fig. 8. Detail of connection of column and diaphragm
Fig. 9. Detail of new foundation under shear walls
10.4 RUMANIAN EXAMPLE ON REPAIR AND STRENGTHENING OF A REINFORCED CONCRETE FRAME STRUCTURE

10.4.1 Case Study

Reinforced concrete frame structure of 33 identical buildings in a housing area in Asuan (Algeria), five-storied (basement + ground floor + 3).

The structure consists of two-way frames, having two 4.80 m bays in one direction and nine 3.00 m bays in the other. The columns have 50x20 cm cross-section dimensions. The transverse beams have a height of 50 cm, the longitudinal ones 30 cm, that is a height-to-span ratio of about 1/10 (Fig. 1a and 1b).

The partitions are of hollow bricks, generally 5 cm thick brittle, very plastered on both faces. The layout is shown in Fig. 1a and 1b (plan and cross-section).

The structural layout is good, the state of stress is clearly determinable. The dimensions of the structural members are reasonable except for the width of the columns in one direction which is obviously too small.

In the 1980 earthquake 5 structures collapsed, 13 suffered heavy damage and 15 medium damage. All collapses occurred in the longitudinal direction as it was to be expected and are due to the complete destruction of the columns at ground story, the upper stories being almost undamaged even after the collapse (Fig. 2).

The damage consisted generally of the complete destruction of the partitions at ground story and their destruction in a proportion of the first story (about 30 - 40%).

The column heads at ground story were destroyed, the concrete crushed, the reinforcement buckled. All kinds of other damages could also be noticed, especially in the columns. Fig. 3 is an example of a representation of the damage pattern. Fig. 4 and 5 show some typical damage.

The cases of the disaster are obviously the very high flexibility in the longitudinal direction, the low steel ratio in the columns, and the very low shear reinforcement (stirrups) of the columns. Here are some figures illustrating these causes:

The mean value of the gravitational load working stress to concrete strength ratio \( n = \frac{\sigma_g}{f'_c} = 0.50 \), about twice as high as recommended.

The story drift at ground story (computed for a base shear force coefficient of about 0.10) is of 1/40 longitudinal and 1/125 transversal. By such values 2nd order effects may be of the same order as the 1st order ones and probably have decisively contributed to the collapse.

Two strengthening concepts have been examined:

Variant 1 - strengthening the existing frames as such, by jacketing of the columns and beams.

Variant 2 - redesigning the structure by introducing new shear walls to take up the entire earthquake action.
A decision was taken in favor of Variant 2, because of:

- technical difficulties in realizing in this case proper frame joints by jacketing by contrast with the technological simplicity and flexibility in layout in the shear wall structure;

- better protection of the nonstructural elements in Variant 2;

- less cost and quantity of work.

The layout of the new shear walls is shown in Fig. 6. Note that the shear walls replace some destroyed infill walls (which would have to be reconstructed in any case), or are located on the outside of the building in order to diminish the intervention on nonstructural elements and finishes.

Construction details concerning the shear walls and their connection with the existing structure are shown in Fig. 7.

The shear walls could be assumed as fixed at ground floor level because the basement is conveniently stiffened by the existing contour walls. Nevertheless, some intervention was necessary even in the basement story, because the external columns and infill walls had been cast in two phases, with no shear connection between them.
Fig. 1

G+3 CROSS SECTION

G+2 CROSS SECTION

480 390

306 306 306 306

240

480 390
10.5- TURKISH REPAIR AND STRENGTHENING CASE STUDY

10.5.1- Introduction

The Adapazarı Earthquake occurred on July 22, 1976 and caused damage to a large number of buildings at the city of Sakarya. Ribbed slab reinforced concrete buildings suffered fairly heavy damage during the earthquake. The Sakarya Government Office Building was one of the severely damaged buildings. Various techniques were used to repair and strengthen the structure. Columns and beams were repaired and strengthened by means of jacketing technique. The most severely damaged columns were demolished and replaced. Furthermore, additional shear walls were provided in order to increase structural stiffness and strength of the structure so as to reduce damage in future earthquakes.

10.5.2- Outline of Earthquake Characteristics

The parameters of the earthquake are as follows:

Name: 1967 Adapazarı Earthquake
Date: 7:20 P.M., July 22, 1967
Epicenter: Latitude 40°.7N, Longitude 30°.8E (On the North Anatolian Fault)
Depth of Focus: 5 - 6 km
Magnitude: 7.2

The seismic intensities were mapped as shown in Fig. 10.5.1. The maximum M.M. intensities were estimated to be IX-X at the epicenter and VII-VIII at the building site which is only 36 km. from the epicenter. The earthquake caused a fault 55 kms long which is the continuation of the 1957 Abant Earthquake. 10-140 cm horizontal and 10-90 cm vertical displacements were observed at the fault. Damage was observed within an area of 7000 km² around the epicenter.

10.5.3- Brief Description of the Building

The Sakarya Government Office Building, as shown in Figs.10.5.2 and 10.5.3, is a five-story reinforced concrete structure. Plan dimensions of the building are 14.20 x 40.00 m. The materials used in construction are ordinary concrete (f_c=16 N/mm²) and mild steel (f_y=220 N/mm²). Ribbed floor slabs consist of 10 x 37 ribs at 50 cm intervals spanning in transverse direction. The beams in both directions are flat with a depth of 37 cm. The dimensions of columns vary between 23 x 30 and 40 x 65. The foundation soil is very poor. The allowable stress was taken as 0.1 N/mm² in the original design and a raft foundation with a slab thickness of 25 cm was constructed. The dimensions of foundation beams are 50 x 150 and 40 x 90 in transverse and longitudinal directions respectively.

10.5.4- Results of Damage Survey

The effects of lateral, vertical and torsional vibrations were observed on the damage pattern of the structure. In order to prevent potential damage of the aftershocks, the structure was shored and braced at the ground floor immediately after the earthquake, as shown in Fig. 10.5.4. Damage to the building caused by the earthquake was massive; however most of the structural damage was concentrated around the staircase and the elevator. Brief description of the significant damage to various elements is as follows:
(1) Most of the columns were slightly or severely damaged. The damage which involves extensive cracking, spalling and crushing of the concrete was especially concentrated to the first and second story columns. Damage to columns was in the form of compression, shear and torsion cracks or failures. Column T4, at first story, completely lost its load bearing capacity. Location of the slightly or severely damaged columns is shown in Fig. 10.5.5.

(2) Damage to beams and ribs was in the form of anchorage failures and concentrated around the staircase and along the column line 1. Location of the damaged beams and ribs is shown in Fig. 10.5.6.

(3) Hollow concrete block masonry in-fill walls failed due to shear. X type shear failures were severe at in-fill walls parallel to the longitudinal direction of the structure.

(4) No damage was recognized at the foundation the permanent settlement at column line 1. Location of the overall severe damages is shown on the elevation in Fig. 10.5.7.

Principal causes of the damage are as follows;

(1) The design seismic load was insufficient. The base shear coefficient was taken as 0.04. This coefficient should be 0.10 according to the new Turkish Code which has become effective since 1975.

(2) Rigidity distribution of the structure was not uniform. The rigidity of the longitudinal direction of the structure was insufficient.

(3) Proportioning and detailing of structural components were inadequate. The column and beam sections were not sufficient and the shear reinforcement was inadequate.

(4) Quality of concrete was rather low, and workmanship was poor.

10.5.5- Repair and Strengthening Methods

The columns were repaired and strengthened by the steps indicated below:

(1) Concrete cover of all columns was removed.

(2) Cement mortar was injected into the cracks.

(3) All columns were jacketed. The thickness of the jacket was estimated in a manner that the area of the added section would be slightly less than the area of the existing column. Increases in column sizes were accomplished by adding reinforcement adjacent to existing column, and by spirals as confining steel. The ends of the vertical reinforcement were anchored into adjacent beams. Jacketing was implemented from the foundation to the roof.

(4) The column T4 was renewed. After having installed temporary supports around the column to hold up the upper floor, the concrete was completely demolished and part of the main reinforcement was removed. Then additional reinforcement was provided and high-early-strength concrete was placed.
After repair and strengthening of the columns, the structure as a whole was also stiffened and strengthened as described below:

(1) Channel shaped shear walls were provided at both short façades of the building, as shown in Fig. 10.5.8. Furthermore, two shear walls were constructed at column lines 8 and 9 in order to eliminate the eccentricity caused by the existing shear walls around the staircase. The ends of the vertical and horizontal reinforcements were anchored into adjacent columns and beams in order to provide an integral wall unit. The wall thickness was so determined that the ultimate strength would not be governed by shear failure of the wall.

(2) In order to achieve sufficient stiffness, strength and structural integrity at the longitudinal direction of the structure, the sections of the beams at the column lines A, T and V were increased, especially at the ground floor and at the roof.

(3) The raft foundation was checked on the basis of the increased weight of the building and the foundation beams along the longitudinal direction of the structure were strengthened by means of jackering.

(4) 67 tons of gravel which existed at the roof for insulation was removed.

General repair and strengthening scheme of the building is shown on the elevation in Fig. 10.5.9.

Characteristics of the structure, before the earthquake and after the repair and strengthening are listed in Table 10.5.1.

10.5.6 - Conclusions

Sakarya Government Office Building which suffered very severe damage during the 1967 Adapazarı Earthquake was repaired and strengthened within the 8 months after the earthquake. Shear walls were provided against future earthquakes. The overall strength of the structure was increased. Particular care was given to detailing of the connections. Replacement of the damaged building would have cost 1.50 times of the original cost but, the repair and strengthening cost only 10% of it.

The consulting engineers [1] who are responsible of both the repair and strengthening project and its application, believe that the structure has been adequately repaired and strengthened against future earthquakes. However, this particular structure can be considered as being somewhat over-strengthened. It would be more practical to keep the intervention at a lower level, hence avoid certain complications related with redistribution and detailing.

10.5.7 - References

[2] "Specifications for Structures to be Built in Disaster Areas"
Turkish Government, Ministry of Reconstruction and Resettlement,
Earthquake Research Institute, July 1975, Ankara.


<table>
<thead>
<tr>
<th>Feature</th>
<th>Characteristics before the earthquake</th>
<th>Characteristics after the repair and strengthening</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiff direction of the structure y direction</td>
<td>Almost equal</td>
<td></td>
</tr>
<tr>
<td>Predominant period of soil</td>
<td>~1.0 sec.</td>
<td>~1.0 sec.</td>
</tr>
<tr>
<td>Natural fundamental periods</td>
<td>Tx≈1.38 sec.</td>
<td>Tx≈0.35 sec.</td>
</tr>
<tr>
<td></td>
<td>Ty≈1.08 sec.</td>
<td>Ty≈0.19 sec.</td>
</tr>
<tr>
<td>Average stress on soil</td>
<td>7.20 t/m²</td>
<td>7.92 t/m²</td>
</tr>
<tr>
<td>Weight of the building</td>
<td>4370 tons.</td>
<td>4732 tons.</td>
</tr>
<tr>
<td>Coordinates of the center of gravity (average of all stories)</td>
<td>x_G=20.18 m</td>
<td>x_G=19.96 m</td>
</tr>
<tr>
<td></td>
<td>y_G=7.15 m</td>
<td>y_G=7.21 m</td>
</tr>
<tr>
<td>Coordinates of the center of rigidity (average of all stories)</td>
<td>x_R=23.75 m</td>
<td>x_R=20.10 m</td>
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<tr>
<td></td>
<td>y_R=8.10 m</td>
<td>y_R=7.13 m</td>
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<td>e_x=8.9 %</td>
<td>e_x=0.4 %</td>
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<tr>
<td></td>
<td>e_y=6.7 %</td>
<td>e_y=0.6 %</td>
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<tr>
<td>Energy absorption capacity</td>
<td>poor</td>
<td>fair</td>
</tr>
<tr>
<td>Base shear coefficient for elastic response</td>
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<tr>
<td>Base shear coefficient for inelastic response</td>
<td>0.06 - 0.08</td>
<td>0.12 - 0.16</td>
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</tbody>
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Table 10.5.1
Fig. 10.5.3

ELEVATION A_A
Fig. 10.5.5
Fig. 10.5.6
10.6 YUGOSLAV EXAMPLE OF REPAIR AND STRENGTHENING OF DAMAGED TOWN HALL BUILDING AT ULCINJ, MONTENEGRO*

10.6.1 Abstract

Presented in this paper is an example of repair and strengthening of the Town Hall building at Ulcinj which was damaged due to the April 15, 1979 earthquake in Montenegro, Yugoslavia.

The example is selected as typical since following detailed structural analysis and studies of the site and the aseismically designed building it has been concluded that the original structural system can sustain seismic loads due to future earthquakes after an adequate repair of the structural elements by injection of epoxide resines into the column cracks and in the repaired masonry infill which suffered partial damage and failure due to local instability effects.

10.6.2 Introduction

The earthquake of April 15, 1979 which struck the Montenegro coastal region in Yugoslavia, is one of the strongest earthquakes which ever occurred within the European continent during this century. Beside the main shock which caused severe destruction and loss of human life, there occurred a series of strong aftershocks during the next two months. These aftershocks increased the damage level and caused permanent psycho-physical torment of the population living in the whole area affected by the earthquake.

The epicenter of the main shock is at lat 41°58' N, 19°00' E, focal depth is \( h = 40 \text{ km} \) and the magnitude \( M = 7.2 \). The earthquake is recorded by more than 50 three-componental accelerographs (SMA-I) and the thirty seismoscopes (WM-I) on a territory of \( 50.000 \text{ km}^2 \).

The devastating earthquake affected not only a great part of the Montenegro coastal area, but also a larger part of the territory of Yugoslavia. Structures constructed in various structural systems and material, erected at diverse places and at different times, suffered damages of different degree and character, which required different economic and structural conditions for their repair. Reinforced concrete structures built during the last ten years according to the existing construction codes are of special interest.

It is necessary to mention that in this large area, various structures can be seen--old structures built in a traditional way, stone buildings, brick buildings strengthened by vertical and horizontal belt courses as well as frame reinforced concrete structures with different in-filling, frame structures strengthened by diaphragms, structures consisting only of diaphragms and

* This example was prepared by Miodrag Velkov, Dr. Scs., Professor, Institute of Earthquake Engineering and Engineering Seismology, Skopje, Yugoslavia, and Predrag Gavrilovic, Dr. Scs., Associate Professor, Institute of Earthquake Engineering and Engineering Seismology, Skopje, Yugoslavia.
large panel prefabricated systems. In this paper, special attention has been
given to a reinforced concrete structure with emphasis on the analysis of
the original and damaged structural systems, investigation of site and soil
conditions, establishment of the seismic criteria and selection of an ap-
propriate method for repair and strengthening.

10.6.3 Description of the Building

The building is a reinforced concrete structure with three stories
above ground level, constructed of bearing frames with infilling of shallow
bricks. The structure is square in plane with a central courtyard. The plan
of the basic structural system and the outline of the structure are shown in
Fig. 1 and Fig. 2.

The built-in material is concrete with crushing strength between $f'c=20-30$ MPa, steel for reinforcement CRB of 40/50. The cross-sections of the
structural elements, columns and beams, with respect to the spans, constitute
a relatively flexible system, in which declined staircase slabs do not prevent
deforation of the system, as floor slabs are suspended on brittle cables.

The structure was founded on solitary footings, or linked solitary
footings, in soil of moderate quality (sandy clay) with ultimate bearing capa-
city ranging from (0.30-0.36) MPa.

10.6.4 Damage of the Building

The basic structural system experienced minor damage, such as cracks
in some column-to-beam sections (more in the columns). The cracks are hardly
larger than 1.00 mm, and are usually in the range of 0.3 to 1.0 mm (fig. 3).
It is obvious from the patterns of damages, in all the three floors of the
structure, that the earthquake caused deformations which induced nonlinear
behavior of some sections of the system often with horizontal cracks in the
concrete. By observation of the major part of cracked sections of the struc-
tural system it was concluded that the damaged sections were in nonlinear
range, and the ultimate strength for deformations was not exhausted. However,
the earthquake caused significant displacement of the structural system, which
caused total collapse, or falling down of brick in-filling, which gave the
first impression of the structure being severely damaged. Soil deformations
or settlement have not been observed. Cracks have been observed off the joints
as well as bending cracks on uniform distances (Fig. 3).

Such structural behavior, and damage degree as observed in situ, ne-
cessitated detailed analysis of the system, so that the results obtained by ana-
lysis of the mathematical model of the structure are in good correlation with
the actual behavior of the structure during the earthquake.

10.6.5 Seismic Design Parameters

The epicentral zone of the April 15, 1979 earthquake was in the sea,
south of Ulcinj, which is relatively close to the site of the structure.

The geotechnical characteristics of the site were determined based on
geological, field and soil mechanical investigations. In this case, by dyna-
ic analysis of the mathematical model on eight horizontal components of ground
acceleration taken at Ulcinj, Bar and Petrovac, applying deconvolution to the
bedrock and normalization to the average level of maximum acceleration, the
amplification factor is determined as DAF \text{ average} = 1.85. For seismic analysis of the structure and possible repair works the seismic effect was determined for a return period of 50 years, \( a_{\text{max}} = 0.24 \text{ g} \), and for a return period of 200 years, \( a_{\text{max}} = 0.42 \text{ g} \). For the April 15, 1979 earthquake the seismic intensity on the ground surface was evaluated as 0.35 g. Records from Ulcinj, Bar and Petrovac earthquake of April 15, 1979 have been used for the structural analysis.

10.6.6 Analysis of the Structural System

Based upon the geometrical characteristics and stiffness and strength characteristics, as well as according to the seismic parameters of the site, a complex seismic analysis of the existing structural system for different seismic intensities and frequency contents and for several variations of the mathematical model of the structure have been performed.

By static analysis for gravity loads (useful load was estimated to be like the one during the earthquake) and the equivalent of the static seismic loads, the distribution and the amounts of the static values of the three-dimensional model in each element separately have been determined. Then, applying the data on the section geometry, the quality of the concrete and reinforcement used, the ultimate bearing capacity and deformations for the characteristic section of each element separately have been determined. By comparison of the ultimate bearing capacity values to the static-equivalent values, the load factors were obtained which enabled estimation of the sequence of occurrence of nonlinear deformations in the structural system, i.e., the probable process of mechanism formation within the system.

Applying the results from the above analysis, the "force-displacement" diagrams were drawn as resultant diagrams of all the members on the floors, for each direction separately and in two variations, and thus the mathematical model of the system was formulated, since it is necessary for the dynamic analysis. Fig. 4 shows graphic presentation of the \( P_x-\Delta_x \) relation for the first floor.

In Fig. 5 the results obtained by analysis are shown graphically. It can be concluded by the analysis that the system has sufficient strength, therefore the scheme diagrams, lateral global force acting at the base of the mathematical model of the system, until yielding, is:

For x-x direction \( P_x = 0.225 \text{ Q} \) \( T_x = 0.56 \text{ sec} \) \( \delta_x = 1.075 \text{ cm} \)

For y-y direction \( P_y = 0.268 \text{ Q} \) \( T_y = 0.59 \text{ sec} \) \( \delta_y = 1.24 \text{ cm} \)

Cracks and yielding in some frame members occur successively, according to the nonlinear deformation mechanism, thus the first cracks in the \( R_1(y-y) \) frame occur for even \( P_{CR} = 0.12 \text{ Q} \) and then cracks in other structural members in the system.

The earthquake effect of different intensity caused nonlinear deformations of the system. The maximum relative deformations for the occurred earthquake (as estimated) were observed at the III floor, \( \delta_{\text{max}} = 3.32 \text{ cm} \), i.e., \( \text{hi}/\delta_{\text{max}} = 96 \), which proves the system to be quite flexible and it suffered considerable deformations which could occur due to earthquakes of such high intensity. However, the required ductility quantities (Figs. 5 and 6) are within the allowable range. It means that the structural members of the system
suffer nonlinear deformations without exceeding its bearing capacity. However, it should be emphasized that the frequency content of the earthquake is such that some members which experienced heavier damage were also influenced by a large number of plastic excursions.

By comparing the analytical results with the degree of structural damage due to the earthquake it can be concluded that:

- The basic structural system exhibited minor damage, i.e., the stresses and strains exceed the yield point in a large number of column-to-beam sections forming the frame structure for all three stories of the structure, however the deformations did not exceed the bearing capacity of the sections and the cracks in the concrete indicate a deformability order below the ultimate one.

- Damage degree of frame beams, though of a lesser order than in the columns, shows also minor deformations as compared to the ultimate state for the concrete in the damaged sections.

- High relative story deformations due to the earthquake being in complete correlation with the analytical results, caused severe damage and failure of partition walls in the structure which made the first impression of the structure being heavily damaged.

10.6.7 Method of Repair and Strengthening

Considering the repair purpose, and according to the analysis of structures, it has been concluded that the existing structural system of frames in both orthogonal directions is the basic system to sustain gravity and seismic forces, since they are of sufficient strength and deformability capacity.

All the damages of the system should be repaired by epoxy injection, following instructions specially prepared for this purpose, while all the partition walls (except those in the sanitary sections) were replaced by light built-in furniture.

Considering the technique for repair of the reinforced concrete structural elements, and according to the size of the cracks, the following principle was applied: the cracks were closed on the surface side by injection of epoxyde emulsion and an additive, added to the epoxydemass, and epoxyde resin was injected under pressure of 2-4 atmospheres, with strict supervision of the execution.

It should be noted that the selection of this repair method was primarily imposed by the damage level of the structural elements (fine bending cracks without shear influence, 0.3 - 1 mm cracks off the frame joints) which determined the level and state of deformations. On the other hand, applying results by tests on earthquake-damaged reinforced concrete elements repaired by epoxydes and epoxyde concrete, stiffness deterioration is evident (in case when failure mechanism occurs in the beams and columns but not in the joints). Hence, repair of structural elements to provide seismic stability of the whole building is of limited value. Should more than 25% of all the elements be repaired, the structure can no longer be considered as original. Therefore, in
addition to epoxyde injection interventions other repair and strengthening measures should be introduced. In the case considered the extent of the repairs is small, and detailed analysis of the deteriorated stiffness has proved that the structural response meets the tolerable limits of displacement and ductility, for both the design earthquake and maximum earthquake levels. Also, the load carrying capacity is of sufficient level. Considering the safety of the existing, damaged and repaired structure based on the deformability and load carrying capacity, and given the seismic parameters for the site, it was concluded that the most economic way of repair and strengthening would be by injection of the cracks of the structural elements and by adding partition walls. In that case it was possible to attain approximately the original stability of the building. Analyses proved that this state complied with the seismic stability criteria.

10.6.8 Conclusions

Structural repair applying epoxydes, epoxyde resins and epoxyde concrete should be used in strictly confined cases, considering the level of knowledge regarding the behavior of the repaired elements as based on experimental and theoretical studies. In general, it should be noted that while epoxyde-repaired elements show favorable characteristics for elastic and static conditions, under cyclic and nonlinear loads rapid stiffness deterioration occurs, thus emphasizing the delicate application of this repair method. Hence, this method can be applied only in cases where the earthquake excitation caused a limited degree of cracking in the structural elements, and where structural analyses prove that such repaired structures can sustain possible future seismic effects. Furthermore, this repair and strengthening method can be applied only when the total volume of the structural elements to be repaired does not exceed 25 percent.
Fig. 1. Plan of the basic structural system

Fig. 2. View of building
Fig. 3. Damage to column

Fig. 4. Summary story $P-\Delta$ diagram
Fig. 5. Story displacement and ductility

1. Capacity yield displacement
2. Ultimate displacement capacity
3. Displacement for design earthquake intensity $a_m = 0.24 \text{ g}$
4. Displacement for max. earthquake intensity $a_m = 0.42 \text{ g}$

Fig. 6. Table of principal results obtained by analysis of capacity of structures and non-linear response analysis

<table>
<thead>
<tr>
<th>Story</th>
<th>Yield story displacement (sm)</th>
<th>Displacement capacity (cm)</th>
<th>Required displacement and ductility factors</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Displacement</td>
<td>Ductility</td>
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Fig. 6. Table of principal results obtained by analysis of capacity of structures and non-linear response analysis
APPENDIX I

GLOSSARIES

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Tension
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Wing wall
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Axial force

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Base Shear coefficient
Beam
Bearing capacity
Belt
Bending
Biaxial
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Bond
Brace
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Buckling

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Ov
Hajlítás
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Merevítő rácsoszás
Rideg
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Kihajlás

Helyszínen készült
Cement
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Oszlop
Nyomás
Beton
Beton takarás
Megszorítás, korlátozás
Kapcsolat
Szerkezet, építmény
Kapcsolati hely
Ellenőrzés
Költség
Repedés
Kritérium, feltétel
Gőrbület
Ciklikus

Károsodás
Károsodási jelleg
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Lehajlás
Alakváltozás
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Ultimate strength
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Working stress

Yield stress

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Reproiectare
Redistribuire
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Armătură
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Structură
Sistem

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Întînderă
Agrafe
Torsione

Rezistență ultimă

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Perete
Plasă de armătură

Sudură
Perete adăugat la un stîlp
Efort unitar

Rezistență de curgere
GLOSSARY
ENGLISH - TURKISH

Analysis
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Construction joint
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Cost
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Tekrarlanan

Damage
Damage pattern
Damping
Deflection
Deformation
Degree
Demolish
Design
Detail
Diaphragm
Displacement
Distribution
Dowel
Drift (interstory)
Ductility

Hasar
Hasar şekli
Sönüm
Sehim
Deformasyon
Derece
Yikma
Tasarlama
Detay-Ayrıntı
Diyafram
Yerdeğiştrime
Dağıtım
Pım
Yatay yerdeğiştrime
Süneklik
Earthquake
Element
Emergency
Energy absorption
Epoxy
Existing
Experiment
Failure
Fatigue
Feasibility study
Flexible
Footing
Force
Foundation
Fracture
Frame
Girder
Grout
Hinge
Hoop
Horizontal
Infill wall
Injection
Intensity
Intervention
Investigation
Jack
Jacket
Joint
Large panel
Level
Load
Masonry
Mass
Material
Measure
Mechanism
Modulus of elasticity
Moment
Mortar
Multi-story
Panel
Period
Plate
Poisson's ratio
Post-earthquake
Precast
Preliminary
Prestressed concrete
Deprem
Eleman
Acil
Enerji yutma
Epoksi
Mevcut
Deney
Göçme
Yorulma
Olabilirlik çalışması
Esnek
Sömel
Kuvvet
Temel
Çatılaak
Çerçeve
Ana kiriş
Şerbetleme
Mafsal
Kolon etriyesi
Yatay
Dolgu duvarı
Enjeksiyon
Şiddet
Müdahale
Araştırmaya
Mantolama
Düğüüm noktası
Büyük pano
Düzey
Yük
Kütle
Malzeme
Onlem
Mekanizma
Elastisite modülü
Moment
Harç
Çok katlı
Pano
Periyod
Levha
Poisson oranı
Deprem sonrası
Hazır
İlk, ön
Ongerilmeli beton
Ratio
Redesign
Redistribution
Reinforced concrete
Repair
Repeat
Replacement
Resistance
Restoration

Safety
Scaffold
Seismic
Settlement
Shear force
Shear modulus
Shear wall
Short
Shotcrete
Slab
Spiral
Steel
Steel net
Steel profile
Stiffness
Stirrup
Stitching dog
Stone
Story
Strength
Strengthening
Stress
Structure
System

Tendon
Tensile strength
Tension
Tie
Torsion

Ultimate strength
Vertical

Wall
Web reinforcement
Wedge
Welding
Wing wall
Working stress

Yield stress

Oran
Yeniden tasarlama
Yeniden dağılım
Betonarmel
Onarım
Tekrar
Yerine koyma, değiştirme
Dayanım, mukavemet
Restorasyon

Güvenlik
Sismik
Çökme
Kesme kuvveti
Kayma modülü
Perde
Kısa
Püskürtme betonu
Düşeme
Pret
Çelik
Çelik ağ
Çelik profil
Rijitlik
Etriye

Kat
Dayanım, mukavemet
Takviye
Gerilme
Yapı
Sistem

Gergi
Çekme dayanımı
Çekme
Etriye
Burulma

Taşıma gücü
Düşey

Duvar
Gödve donatısı

Kaynak
Kanat duvarı
Çalışma gerilmesi

Akma gerilmesi
GLOSSARY
ENGLISH - SERBOCROATIAN

Analysis  Analiza
Anchorage  Ankerovanje
Axial force  Aksijalna sila
Bar  Sipka
Base Shear Coefficient  Koeficijent smicanja u osnovi
Beam  Greda
Bearing capacity  Kapacitet nosivosti
Belt  Serklaz
Bending  Savijanje
Biaxial  Biaksijalnan
Bolt  Veza
Bond  Razupirac
Brace  Krt
Brick  Opeka
Buckling  Izvijanje
Cast-in-place  Lijen na licu mesta
Cement  Cement
Collapse  Rusenje
Column  Stub
Compression  Pritisak
Concrete  Beton
Concrete cover  Zastitni sloj betona
Confinement  Obavijenost (utegnutost)
Connection  Veza
Construction  Gradjenje
Construction joint  Konstruktivan veza (cvor)
Control  Kontrola
Cost  Kostanje
Crack  Pukotina
Criteria  Kriterija
Curvature  Kriva
Cyclic  Ciklus
Damage  Ostećenje
Damage pattern  Nacin ostećenja
Damping  Prigusenje
Deflection  Deformacija
Deformation  Deformacija
Degree  Stepen
Demolish  Rusenje
Design  Projekt
Detail  Detalj
Diaphragm  Dijafagma
Displacement  Pomeranje
Distribution  Rasporedjenost
Dowel  Klin, cep
Drift (interstory)  Spratno pomerajne
Ductility  Duktilnost (zilavost)
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