SEISMIC UPGRADING OF EXISTING STRUCTURES

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ABSTRACT

The seismic upgrading of existing structures is a growing concern for engineers in Canada. There is very little guidance in terms of technical literature or code guidelines for design engineers faced with the problem of upgrading an existing structure which is deficient with respect to seismic resistance. This thesis attempts to provide useful information for the seismic upgrading of existing structures. Code approaches used or under study in other countries are described. Case studies of structures which were repaired and strengthened in Mexico City after the 1985 earthquake and case studies of structures which were upgraded in Canada are presented. Design problems that are particularly associated with seismic upgrading are discussed and a brief survey of recent experimental findings is made.

RESUME

La réhabilitation sismique des structures existantes est une préoccupation croissante pour les ingénieurs Canadiens. Il existe très peu d'information sous forme de littérature technique ou de procédure règlementaire pour l'ingénieur en structure qui doit faire face au problème de la réhabilitation d'une structure existante qui est déficiente du point de vue de la résistance sismique. Cette thèse tente de procurer d'utiles informations pour le renforcement sismique des structures existantes. Différentes approches quant à la règlementation qui sont utilisées ou étudiées dans diverses pays sont décrites. Des études de cas de structures réparées et renforcées dans la ville de Mexico à la suite du tremblement de terre de 1985 ainsi que de structures réhabilitées au Canada sont présentées. Des problèmes associés à la conception des techniques de renforcement sismique sont discuttés et les résultats pertinents d'études expérimentales récentes sont brièvement présentés.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS

A_s	area of tension tie reinforcement
$E_{c,eff}$	reduced initial tangent modulus of elasticity of concrete
E_{cs}	secant modulus of elasticity of concrete
E_o	basic seismic reserve index
E_{oi}	basic seismic reserve index for the i^{th} storey
f_c	stress in concrete
f'_c	compressive strength of concrete
f_{ci}	initial stress in concrete
f_y	yield stress of reinforcement
F_i	ductility index for the i^{th} storey
g	gravitational acceleration
K	ductility factor used in the calculation of the base shear
M_f	factored bending moment
M_r	factored bending moment capacity
M_s	service bending moment
OP	number of occupants
P_f	factored axial load
P_r	factored axial load capacity
P_s	service axial load
Q	ductility factor used in the calculation of the base shear in Mexico
Q_{ei}	elastic response shear force for the i^{th} storey
Q_{yi}	shear yield strength of the i^th storey
r_c	earthquake capacity ratio
t_x	time limit for the upgrading of existing structures
V_{ei}	shear force corresponding to the equal energy criterion (Newmark)
V_{yi}	shear force corresponding to yield
$\epsilon'_{c,eff}$	reduced peak strain
ϵ_{cf}	strain causing stress in concrete
ϵ_{co}	creep strain in concrete
ϵ_{sh}	shrinkage strain in concrete
ϕ_c	resistance factor for concrete ($\phi_c = 0.60$)
ϕ_s	resistance factor for reinforcement ($\phi_s = 0.85$)
μ	ductility ratio

CHAPTER 1

GUIDANCE FOR SEISMIC UPGRADING

1.1 Reasons for Upgrading

When faced with the evaluation of an existing building, a number of options are available, including:

(a) leave the building as is,

(b) totally upgrade the structure to meet the current code provisions,

(c) partially upgrade the structure, or

(d) demolish the building.

The total assessment of an existing building involves many aspects. It requires studies of the structural adequacy, the architectural concept, the integration of the services (mechanical and electrical) and the financial aspects. It must be appreciated that in choosing a course of action for upgrading, significant compromises are usually necessary between the structural and architectural designs[20,21].

The change of use, the expansion, or significant alteration of an existing building requires a complete evaluation of the structure by an engineer. In the seismic evaluation, strength, stiffness and ductility evaluations must be included in the assessment of the safety, the serviceability and the capacity to absorb energy. It should be recognized that structures designed by earlier codes may have serious deficiencies, particularly with regard to the seismic design requirements which have changed substantially over the years. It is assumed that, in general, buildings designed prior to the development of the first earthquake design provisions are potentially more hazardous than those designed after the development of these codes. In this regard, older multi-storey unreinforced masonry buildings are considered to be the most hazardous type of construction. It is important to appreciate that a very high percentage of all existing structures, even those designed and constructed recently, would not satisfy all of the current code provisions. For example, there are many reinforced concrete frame structures which were designed for reduced force levels without the appropriate design and detailing considerations to match the necessary levels of ductility. Also, some buildings have suffered a reduction in their original seismic safety due to events such as; deterioration of materials (e.g., fire damage, corrosion, etc.), foundation settlement, alterations that have weakened structural elements and major changes of use that result in larger force levels (i.e., the use of an existing building for large storage loads).

There is a separate class of buildings called post-disaster facilities which includes hospitals, schools, telecommunication facilities, and strategic defense facilities. These buildings must be operational after major earthquakes, and for this reason, current codes provide additional safety provisions as well as serviceability requirements for the design of these buildings. Because of their importance, the structural adequacy of existing post-disaster buildings should be assessed and, if necessary, upgraded.

1.2 Economic Considerations

In assessing the economics involved in upgrading an existing building, the following factors must be considered:

(a) Upgrading an existing building avoids the cost of demolition and rebuilding.

(b) Careful planning and execution of the upgrading operations may allow continuous revenue if the use of the existing facility is uninterrupted. In contrast, there is no revenue from the use of the facility during the period for demolishing and rebuilding.

(c) The replacement of mechanical and electrical services when upgrading an existing building is typically more costly than the installation of those services in a new building.

(d) By its very nature, strengthening an existing building is labour intensive, requires skilled construction personnel and constant on-site supervision by an engineer. Therefore, the unit cost of placing materials can be many times higher than the unit cost for new construction.

In 1985, building officials in Long Beach, California[16] estimated that the structural work involved in the strengthening of unreinforced masonry buildings, up to the level of the 1970 Uniform Building Code, costs between $15 to $20 per square foot. Other multi-storey buildings cost about $25 per square foot for structural work with the total cost (including structural, architectural and services) being about $45 to $70 per square foot.

Upgrading costs may be justified by economic benefits such as an increases of market value, longer anticipated lifetime, improved expected revenue, and possible tax or depreciation benefits. Building officials in the city of Sebastopol, California[16] reported that, after upgrading and renovation, general and fire insurance rates dropped an average of 50% to 60%, and the real dollar value of the properties tripled. In these cases, taxation was unaffected. In the case of similar adjacent buildings, significant savings resulted if neighboring building owners chose to upgrade at the same time.

In 1981 the Federal Government of the United States[16] introduced a tax incentive programme in an attempt to encourage the restoration and retrofitting of existing structures of historical importance. Such government programmes play an important role in creating the necessary economic incentives to tip the scales in favour of upgrading.

The repair and upgrading of structures in Mexico City to meet the requirements of the 1985 Emergency Code Regulations[11] provide an interesting case of how regulations favoured upgrading. Because of the considerable damage to buildings having heights between 6 and 20 storeys, the height of new construction in the zone of severe damage is now limited to 4 storeys. This height restriction for new construction, together with the relatively low cost of labour, has resulted in a significant number of multi-storey buildings that are being upgraded or repaired rather than replaced by new structures of only 4 storeys in height.

It is imperative that the options of demolishing and rebuilding or upgrading and renovating be thoroughly investigated with detailed economic evaluations. In these evaluations, alternative methods of upgrading may be considered.

1.3 Application of Building Code Requirements

When faced with the evaluation of an existing building, the design engineer has to address a number of difficult problems including the following:

(a) The prevailing building codes mainly address the design and construction of new buildings. No specific guidance is given on how to evaluate or upgrade an existing building.

(b) Some codes (e.g., National Building Code of Canada[27]) adopt an "all or nothing" approach to seismic upgrading. These codes would require full compliance with the latest code provisions whenever structural alterations are being made to an existing building, whereas no upgrading would be required if no changes are being made to an existing building. In many jurisdictions, there is no intermediate level of upgrading permitted. The design engineer must bear the responsibility of interpreting the intent of the code provisions and choosing between the costly solution of full upgrading or taking the risk of doing nothing.

(c) Values of base shears obtained from different editions of building codes can vary widely. Historically, as our knowledge of building response, ground motion, and experience with performance of different types of buildings increased, the base shear demand stipulated in codes changed.

(d) Current codes do not provide criteria by which the performance of older buildings can be evaluated. It is extremely difficult to provide a seismic assessment of an older building which may not conform with the detailed design provisions of existing codes.

Early recognition of these problems led to numerous studies on the appropriate methods for the evaluation of seismic performance in existing buildings and for the upgrading of seriously deficient structures. The different approaches to seismic upgrading can be classified in the following five categories:

(a) Mandatory upgrading: Many people view the implementation of "retroactive" upgrading requirements as unfair in the sense that it is like changing the rules of the game after the game has been played[28]. In addition, mandatory retroactive upgrading has drastic economic consequences and may even have significant political implications. However, it may be an appropriate solution for reducing the seismic risk of post-disaster structures, such as hospitals and schools, particularly in severe seismic regions. In situations such as these, the high cost of upgrading would be politically acceptable because of the perceived need for these measures and because the cost would be borne primarily by the public sector.

(b) The "all or nothing" approach: This approach is described above and is the most common among current building codes.

(c) The compromise approach: Earlier codes in the United States adopted rules which did not impose directly but triggered indirectly the upgrading of deficient structures. This was an attempt to compromise between the mandatory and the "all or nothing" approaches. One such attempt, known as the "25-50% Rule"[16], is described as follows:

"The 25-50% Rule has been the most common triggering method in model codes. Although varying slightly from code to code, the basic rule states that if work is done on a building that exceeds 50% of its value, full compliance with code is required; work with a value between 25-50% of the building has lesser requirements, either negotiated or concerned only with the alterations themselves; work with less than 25% of the value of the building typically only must not endanger public safety or extend existing hazards."[16]

In retrospect, this approach has not been successful in improving the level of public safety since it does not address the potential seismic hazard of the building. For example, a building with a structural system which is known to be hazardous would not be significantly upgraded unless a major renovation job is planned.

(d) The risk evaluation approach: A more rational approach is to base the level of upgrading of deficient structures on an analysis which assesses both the likely performance of the structure and the potential risk for loss of life. Consequently performance criteria less then that for new buildings may be acceptable. This concept was adopted in the recommendations of the Applied Technology Council (ATC[2]) for the development of seismic regulations for existing buildings. In this approach, the seismic performance is described by the earthquake capacity ratio, r_c, which is defined as "the ratio of the capacity of the existing building to resist the effects of earthquake motions to the required capacity for new buildings...". For post-disaster buildings in high seismic zones (i.e., seismic performance category D), the minimum acceptable value of r_c is 0.5. For these structures having r_c less than 0.5, full upgrading to a level of r_c equal to 1.0 is required.

For other structures in moderate or high seismic zones (i.e., seismic performance category C), both the structural adequacy (r_c) and the potential loss of life are taken into account. The risk to human life is accounted for by determining the number of occupants in the building. The minimum acceptable value of r_c is given by the following equation:

$$r_c = 0.25\left(1 + \frac{OP - 100}{700}\right) \qquad (1\text{-}1)$$

but need not be less than 0.25
and cannot exceed 0.50

where r_c = earthquake capacity ratio
OP = number of occupants.

Fig. 1.1 illustrates how the regulations account for these two parameters in assessing the degree of upgrading that is required. It is noted that the determination of the number of occupants in a building has been codified by ATC by specifying the area in

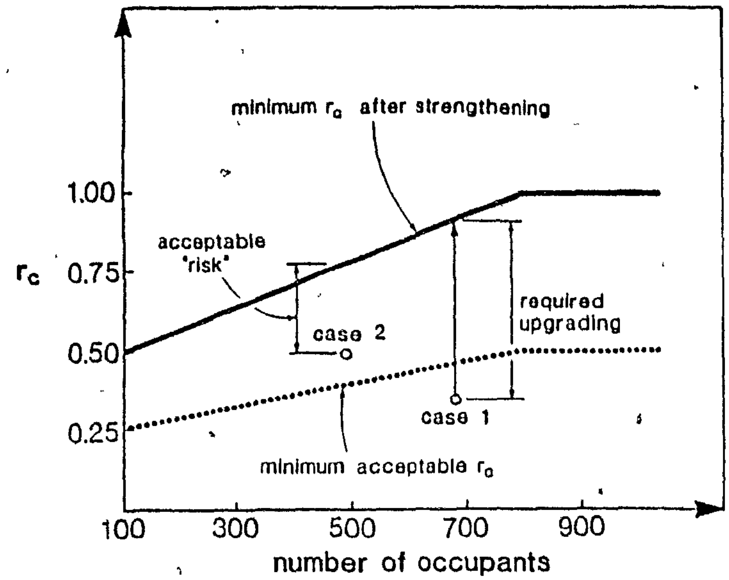


Figure 1.1 Minimum Acceptable r_c (taken from Reference 2).

square feet per occupant for different types of building uses. The dotted line indicates the minimum acceptable value of r_c as a function of the number of occupants. If an existing structure (case 1 in Fig. 1.1) has a value of r_c below this dotted line, upgrading would be required to bring r_c up to the minimum level after upgrading indicated by the solid line in Fig. 1.1. On the other hand, if an existing structure (case 2 in Fig. 1.1) has a value of r_c above the dotted line, then no upgrading would be required. In such a case, the capacity ratio is considered to represent an acceptable risk. It should be noted that the required level of upgrading ranges from $r_c = 0.5$ for low occupancy structures to $r_c = 1.0$ for high occupancy structures thus reflecting the relative risk levels.

The determination of the earthquake capacity ratio, r_c, for an existing structure requires inspection, a considerable amount of engineering judgment and detailed calculations. If the ATC recommendations are translated into the terminology of the Canadian codes, then for each member, ratios of the factored resistance to the factored load is determined for moment, shear and axial load (that is M_f/M_r, V_f/V_r and P_f/P_r). If all of these ratios for all members of the structure are equal to or greater than unity, the building meets the current code requirements and need not be considered for upgrading of the structural system. But if any of these ratios is smaller than unity, then the need for structural upgrading must be investigated. Using the smallest value of r_c obtained, the need and level of upgrading can be determined from Fig. 1.1.

In determining the factored resistances, the construction practices prevailing at the time of construction of the building must be taken into account. Since over the years many improvements have been made, particularly in the area of detailing, it is important that the factored resistances be reduced from that of new construction to account for the lower capacity of older construction. Since concrete structures are particularly susceptible to shear failures in earthquakes, it is important to assign significant reductions for the shear resistance. Additional capacity reductions may be necessary for the other types of construction. An important aspect of this assessment procedure is the judgment of the engineer to assess the actual conditions of the structure and to determine appropriate factored resistances. Judgment is also required in assigning an appropriate seismic response modification factor (i.e., the K factor in the NBC and UBC codes and the R factor in the ATC 3 provisions) when determining the required factored resistances. Because the construction used in many existing structures does not conform with the construction types described in new codes, it is important that the engineer be conservative when determining the seismic response modification factor.

The discussion above related to the determination of r_c for strength evaluation. It is also necessary to evaluate the drift performance of the structure. Separate earthquake capacity ratios, r_c, are determined for each level of the structure. This ratio is defined as the allowable drift divided by the computed earthquake drift. If the smallest r_c determined for drift is less than the smallest r_c determined for strength, then the drift ratio r_c will govern the assessment and would be used to represent the structure in Fig. 1.1. It is noted that the drift is calculated by taking the predicted elastic drift and multiplying by an amplification factor to account for the inelastic deformations.

It is interesting that the ATC regulations address the difficult problem of determining the time limit for carrying out the necessary upgrading. For post-disaster structures in high seismic zones (Category D), the time in years, t_x, permitted is:

$$t_x = \alpha_t r_c \tag{1-2}$$

but need not be less than 1 year
and cannot exceed 15 years.

For other structures in moderate to high seismic zones:

$$t_x = \alpha_t \left(1 + \frac{200}{OP}\right) \tag{1-3}$$

but need not be less than 2 years
and cannot exceed 15 years

where t_x = time limit
α_t = undefined parameter
OP = number of occupants
r_c = earthquake capacity ratio.

The value of α_t has not been defined and should be determined by the local regulatory jurisdiction. The ATC provisions give an example with α_t equal to 12. Fig. 1.2 illustrates the factors influencing the time limit to either strengthen or demolish the building. As can be seen from Fig. 1.2, post-disaster buildings (category D) in high seismic zones would have the shortest time limit. In addition, this approach would satisfy the need to quickly upgrade the most hazardous structures (i.e., having low values of r_c) while also recognizing the need for more rapid upgrading of structures having higher occupancy levels.

(e) The ductility evaluation approach: The response of a building is influenced by a number of factors and can hardly be characterized by a single quantity. The method

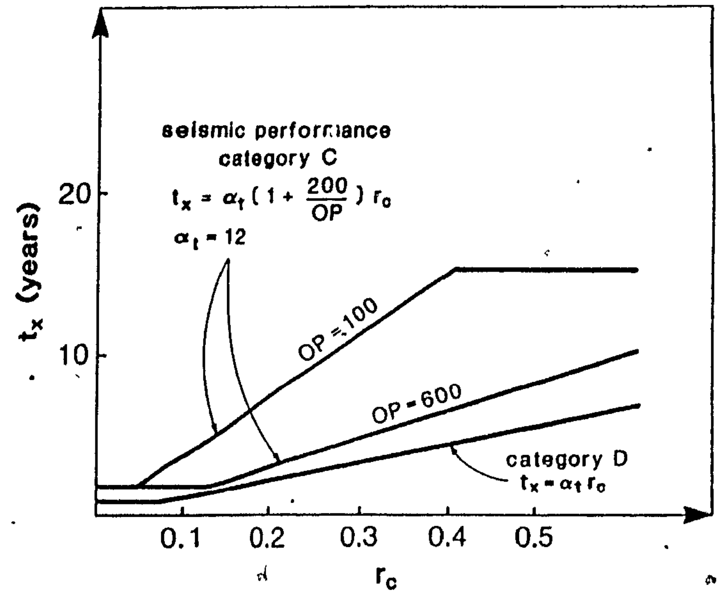


Figure 1.2 Time Limit for the Upgrading of Existing Structures[2].

used to evaluate an existing structure must take into account strength, ductility and energy dissipation for reversed cyclic loading. A quantity termed the basic seismic reserve index[15], E_o, has the following form:

$$E_o = (\text{index for strength})(\text{index for ductility})$$

In this approach, the seismic index E_o is compared with the desirable value of E_o for a structure. Desirable values of E_o are based on experience on the performance of framed buildings in earthquakes. It is assumed that the response is controlled by the characteristics of the columns and that the energy dissipation capacity of the beams are less critical than would be the case if the response were controlled by yielding of the beams.

For a multi-storey building, each storey is likely to have different strength and ductility characteristics so that the E_o value for each storey may differ. Further, the applied lateral shear force for each storey may differ depending on the earthquake motion and the geometry of the building (height, form, etc.). Thus, the different E_o

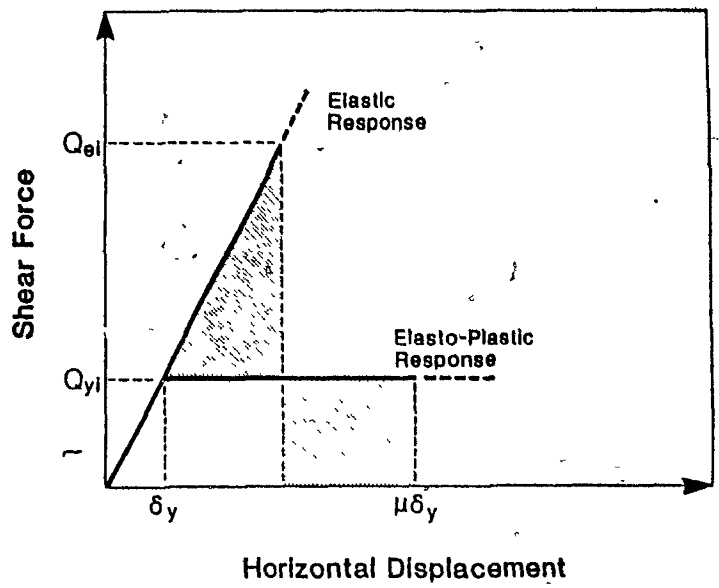


Figure 1.3 Elastic and Elasto-plastic response Equivalence.

values for each storey are defined as follows:

$$E_{oi} = \left(\frac{Q_{yi}}{Q_{ei}}\right) F_i \tag{1-4}$$

where E_{oi} = basic seismic reserve index for the i^{th} storey
Q_{yi} = shear yield strength for the i^{th} storey
Q_{ei} = elastic response shear force for the i^{th} storey
F_i = ductility index for the i^{th} storey.

The ductility index, F_i, is a function of the deflection ductility factor, μ. If the building is assumed to be elasto-plastic and Newmark's equal energy criterion used, then by equating the shaded areas of Fig. 1.2 it follows that:

$$\left(\frac{V_{ei}}{V_{yi}}\right) = \sqrt{2\mu - 1} \tag{1-5}$$

and thus

$$F_i = \sqrt{2\mu - 1} \tag{1-6}$$

Table 1.1 Recommended F_i Values[15].

	MEMBER TYPE	F_i
1	Column with clear height-to-depth ratio less than 2.0 and shear capacity less than flexural capacity (brittle failure).	0.8
2	Column with shear capacity less than flexural capacity but H/d > 2.0.	1.0
3	Column with flexural capacity greater than shear capacity but inadequate hoops and high axial stress.	1.5
4	Column as in 3 and with hoops satisfying code requirements for shear.	2.5
5	Column as in 3 and with hoops satisfying special confinement requirements adjacent to connections.	3.5

F_i values for columns framing panels with brick infill should be multiplied by 0.9.

The F_i values may have to be reduced if the column response is not controlled by flexural yielding at its ends. Recommended values of F_i are shown in Table 1.1.

The storey where the minimum Q_{yi}/Q_{ei} value occurs is likely to be the weakest storey where severe damage would occur in the event of an earthquake. In general, the greater the smallest value of E_{oi} for a frame, the greater the earthquake resistance of the structure. Limiting minimum values of E_{oi} based on Chinese experience from analysis of buildings damaged in the 1975 Haicheng and 1976 Tangshan earthquakes are shown in Table 1.2. More details on the evaluation of the elastic response shear Q_{ei} and the shear yield strength Q_{yi} are given in reference 15. This procedure is used in China and is recommended as a screening procedure to determine if the capacity of a building is questionable.

Table 1.2 E_o for Varying Degrees of Damage[15].

E_o Value	0.4	0.5-0.6	0.7-0.8	0.9-1.0
Damage Expected	Collapse	Severe	Moderate	Slight

1.4 Legal Considerations

When buildings are strengthened to a selected level of performance which do not achieve full compliance with current code requirements for new construction, the responsibility of the level of strengthening should be shared between the owner and the engineer. The proposed criteria and strengthening scheme should be reviewed in detail with the owner, as the strengthening to force levels less then that required by current codes is most likely saving the owner considerable money while increasing the level of security of the building. The owner should share in this criteria decision and understand that his investment is not a guarantee to a damage-free building and that in the event of an earthquake, the building may suffer greater damage than a structure strengthened to full compliance with current codes. The engineer should clearly explain the alternatives and his opinions of anticipated performance so that the owner can intelligently share the responsibility of the decision with the professionals as well as the consequences. When strengthening is the result of a voluntary action and not dictated by regulations, the local building official will usually be agreeable to the selected approach, although he should be contacted for concurrence.

The status of the latest version of the National Building Code of Canada as "The Standard" for the entire country carries considerable weight in legal proceedings. Throughout the code it is made clear that the provisions are "minimum requirements" for an "acceptable level of public safety". Problems arise due to the considerable lag-time of many municipalities in adopting, in their by-laws, new versions of the National Building Code of Canada.

1.5 Construction Execution

Clear drawings and precise specifications for seismic strengthening is essential since the strengthening elements are retrofitted into an existing structure[31]. This requirement is emphasized by the uncertain conditions of strengthening projects. The exact "as-built" conditions are not always accurately known until exposure during the construction work. During construction, it is often necessary to modify details which have been previously prepared on the basis of inaccurate informations. Consequently, constant monitoring of existing conditions is essential. The design engineer must periodically visit the structure during construction in order to view the exposed conditions for damage or distress and to ensure compliance of the construction work with drawings and specifications. Strengthening projects also involve the use of new or seldom used construction materials and techniques which require special instructions to the workmen as well as vigilant inspection procedures. In many situations where the strengthening involves fastening new materials to old existing materials, the quality of workmanship is a crucial factor in the performance of the strengthened structure. Thus, extensive field testing of connectors and fastenings may be necessary.

CHAPTER 2

DEFICIENCIES AND STRENGTHENING MEASURES

2.1 Identification and Evaluation of Structures for Seismic Upgrading

Because of the significant changes that have occurred in the design codes, one of the most important criteria for the identification of deficient structures is the year of construction[3]. Observed damage from previous earthquakes clearly indicates the vulnerability of certain types of construction such as unreinforced masonry and non-ductile reinforced concrete frames. Therefore another important criterion for the identification of possible deficiencies is the type of construction. A third important criterion is the seismic exposure (i.e., seismic zone). Other general characteristics which would be accounted for in a preliminary evaluation include building use, building size, number of occupants, current building condition and the geometry of the lateral load resisting system.

Because the type of construction plays such a dominant role in the performance of the structure, the types of construction known to be vulnerable to severe structural damage are described in the following sections. A summary of common structural deficiencies and possible strengthening measures is given in Table 2-1.

Table 2.1 Common Structural Deficiencies and Strengthening Measures.

TYPES OF STRUCTURES	DEFICIENCIES	STRENGTHENING MEASURES
Unreinforced masonry (non-ductile)	- low shear resistance of walls - poor connections of diaphragms to walls - inadequate reinforcement in diaphragms	- add reinforced concrete walls or steel braces - add ties between diaphragms and walls - reinforce diaphragm
Non-ductile Moment-Resisting Frames	- weak columns, strong beams - low joint shear resistance - may be susceptable to P-delta effects - slabs without beams susceptable to punching shear	- increase column sizes or add walls or braces - same as above - same as above - add walls or braces
Other Types of Structural Deficiencies	- soft storey - large torsional eccentricity - insufficient lateral stiffness - susceptibility of short columns to shear failure	- add walls or braces - add walls or braces to reduce eccentricity - stiffen with shear walls or braces - strengthen column or remove short column effect

2.2 Unreinforced Masonry

Unreinforced and lightly reinforced masonry buildings are probably the largest contributors to the inventory of seismically deficient structures. Typically, they consist of stone, brick or concrete block walls. Older unreinforced masonry structures typically have timber floors with timber or steel interior columns. Deficiencies and failures that have been observed in earthquakes with this form of construction include:

(a) Poor connection between timber floor diaphragms and masonry walls has led to the loss of vertical support for floors as well as the loss of lateral support for masonry walls.

(b) Old timber flooring is typically unable to provide the necessary diaphragm action due to poor connection between the flooring elements.

(c) Inertial forces on the masonry walls, acting perpendicular to the walls, have caused failures of these walls in bending about their weak axis. Some of these severely damaged walls exhibited horizontal cracks in relatively high walls and vertical cracks in relatively long walls.

(d) Walls may fail due to shear forces in the plane of the wall.

(e) Some failures of unreinforced masonry walls have been attributed to deterioration of the walls due to progressive cracking from previous earthquakes and from differential foundation settlement[30].

(f) Because these structures rarely have adequate separation from adjacent buildings, damaged has occurred due to pounding between adjacent structures. In some cases, two very different structures are joined by a party wall which could cause severe damage at their interconnection.

(g) Unsymmetrical layout of masonry walls and large openings in these walls can produce significant torsional eccentricities.

Typically the seismic upgrading of these older masonry structures involves the addition of a supplementary lateral load resisting system and improving the connection between the main structural components, while preserving the masonry exterior. Reinforced concrete shear walls are typically added on the periphery of the building, inside the exterior wall. The advantages of placing the added shear walls on the perimeter of the building are listed below:

(a) The new shear resisting elements, if attached to the existing load bearing system, will also provide integrity of the existing masonry wall.

(b) Locating the elements on the periphery is the most efficient way of improving the torsional resistance and reducing the torsional eccentricity.

(c) The exterior of the structure is more easily accessible during construction.

(d) Interference with mechanical and electrical services is kept to a minimum.

(e) It may be possible to keep the building in operation during construction.

(f) The addition of elements on the exterior face of a structure reduces the need to cut

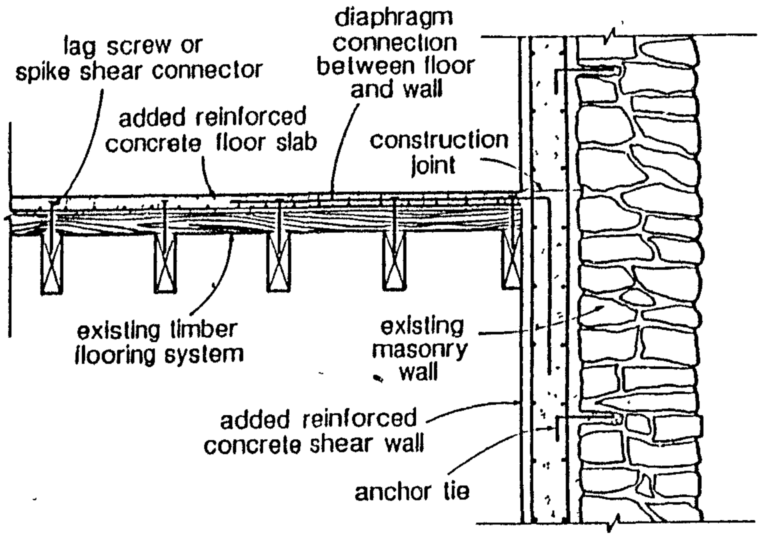


Figure 2.1 Cross-Section of Strengthening Measure for an Unreinforced Masonry Wall.

through existing floors and enables better continuity of reinforcement and the concrete over the height of the structure.

Figure 2.1 illustrates the details of upgrading an unreinforced masonry structure. The first step in the construction is to remove a portion of the flooring adjacent to the masonry wall and then casting a reinforced concrete shear wall against the existing masonry. These added reinforced concrete shear walls may be full length walls or short lengths of walls. The addition of short lengths of walls minimizes interference with existing openings. The placement of short reinforced concrete walls in the corners of the building provides an effective means of tying the walls together. Reinforcing bars anchored into the existing masonry wall act as ties and shear connectors between the two walls. A thin concrete topping, reinforced with welded wire fabric, is made composite with the existing timber flooring by using lag screws or spike shear connectors. This composite floor system serves as a diaphragm and is connected to the shear wall by reinforcing bars which act as both ties and shear connectors between the slab and the wall.

In regions where there is no added concrete shear wall, the floor diaphragm needs

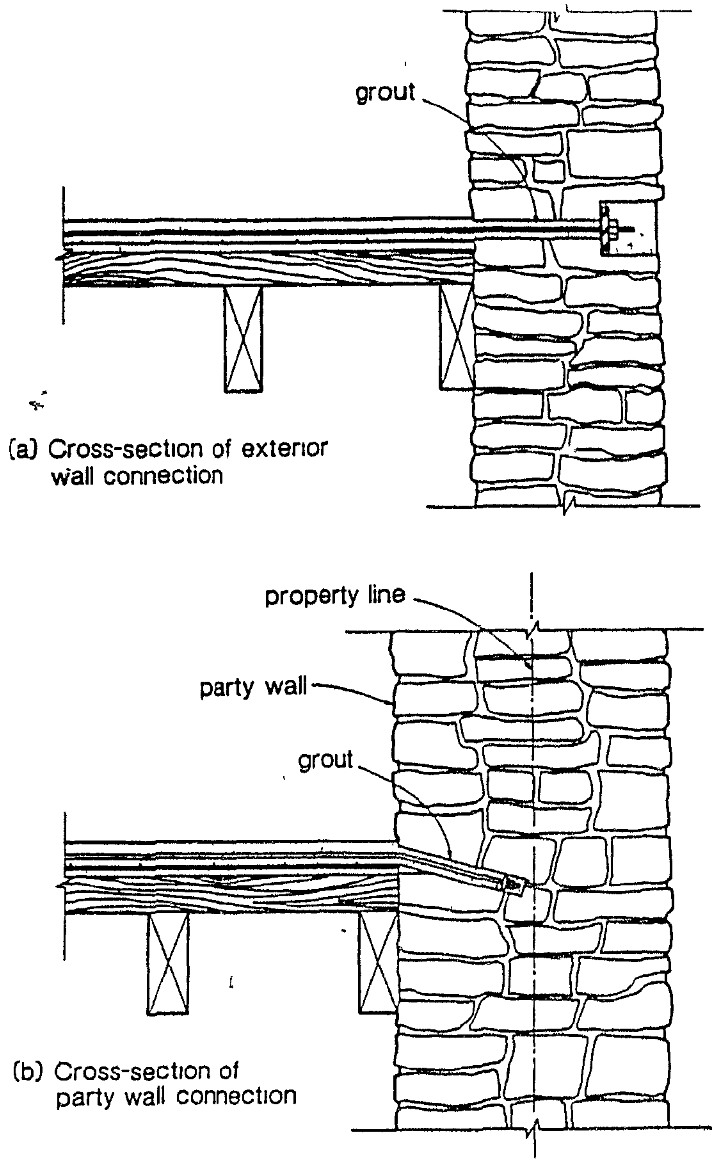


Figure 2.2 Cross-Section of Anchorage Details of a Concrete Floor Diaphragm to a Masonry Wall[4].

to be connected to the masonry wall using connection details such as those shown in Fig. 2.2. A bolt and anchor plate may be used if the exterior of the wall is accessible (see Fig. 2.2a), whereas an embedded anchor may be used for other circumstances (see Fig. 2.2b). The reinforcement between the floor diaphragm and the masonry wall also serves to provide stability to the wall against out-of-plane forces.

An alternative method to strengthen the diaphragm is the addition of plywood flooring. While the sheathing itself is often adequate to develop the necessary design shears, the nailing available to transmit this shear is critical. If the old flooring is removed, then the addition of timber blocking beneath the plywood joints is necessary to provide nailing surfaces between the floor joists. If splitting of the old, dry frame members is a concern, box nails and staples may be used. Pre-drilling of the nail holes, even though costly, may be advisable to avoid splitting. Many older structures have diaphragms consisting of straight-laid boards attached to the joists by nails that may not comply with current code requirements. For these cases, it may be necessary to install a plywood or diagonal-sheathed overlay designed to transmit the entire calculated shear as a new diaphragm. The plywood may be laid with its face grain at a 45 degree angle to the direction of the boards. It is important to ensure that all plywood edges are backed by a nailing surface. Care needs to be taken to ensure that the edges of the plywood do not coincide with the joints in the original flooring. When overlay plywood sheathing is installed, joints should be staggered.

Special attention needs to be given to the detailing of the anchorage of the floor diaphragm to the shear wall. In many existing structures with masonry shear walls, anchorage is provided by edge nailing of the floor decking to a timber ledger which in turn is bolted to the wall. Studies of damage to such structures in recent earthquakes indicate that this is a poor method of anchorage. Such anchorage should be improved by the addition of a bolted steel-strap tie as shown in Fig. 2.3 . The added tie is fastened to the wall through the use of a mechanical anchorage device grouted into a wall cavity or alternatively one of the connections details shown in Fig. 2.2a and 2.2b can be used. The tie is connected to the floor assembly by bolting a metal strap to the floor joist. Additional nailing should be provided to the sheath to which the tie is attached in order to aid in the transfer of the connection forces from the wall into the floor diaphragm. In some cases, new ties may be installed using washer plates on the exterior face of the wall (see Fig. 2.3). One very easy way of providing connection

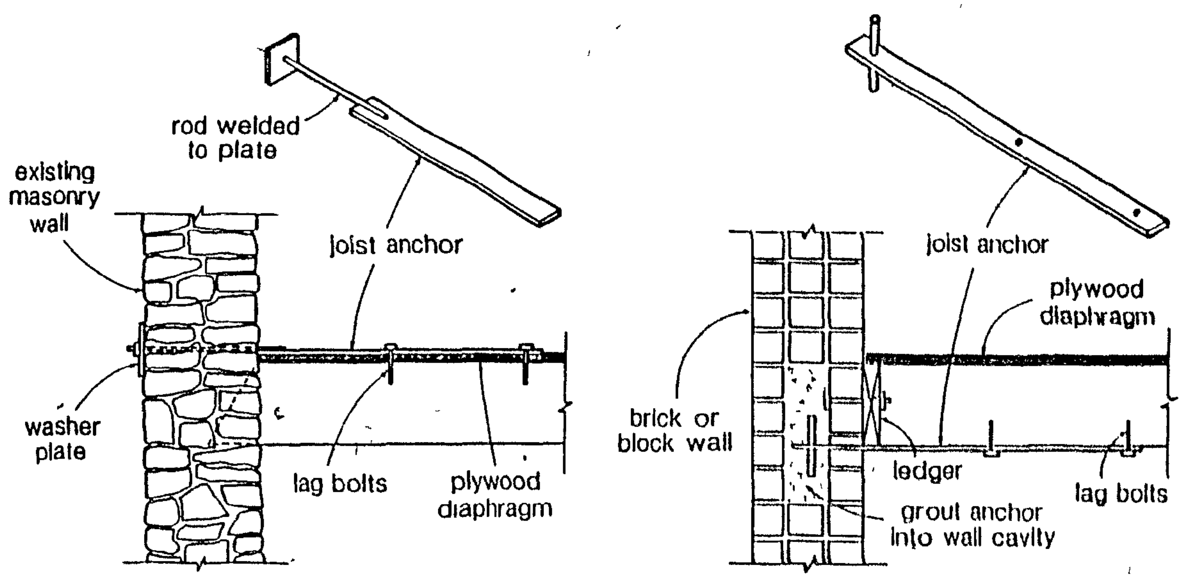


Figure 2.3 Cross-Section of Anchorage Details of a Plywood Floor Diaphragm to a Masonry Wall[13].

between the diaphragm and the walls is to provide a tension tie from one side of the structure to the other. This can be provided by using prestressing tendons below the floor level and anchored on the exterior of the masonry walls.

Construction techniques that have been used in the upgrading and repair of masonry shear walls are listed below:

(a) Shotcreting of concrete has an advantage over conventionally placed concrete by reducing the need for formwork and by reducing the hydrostatic pressures induced on the masonry wall.

(b) Vertical post-tensioning of masonry walls with high strength bars inserted into vertically drilled cores and anchored into the foundation increases the cracking load, the shear resistance and ties the wall together.

(c) Thin concrete overlays with reinforcing mesh have been used in some countries in order to strengthen existing masonry walls. Recent developments of glass-fibre reinforced cement has led to its use in thin concrete overlays.

(d) Cracks and other cavities in existing masonry walls have been repaired with non-shrinking grout, epoxies and special foaming adhesives.

2.3 Non-Ductile Reinforced Concrete Frames

Past earthquakes in Alaska (1964), Caracas (1967), San Fernando (1971), Managua (1972) and Mexico City (1985) have provided many valuable lessons in pointing out the deficiencies in the design and construction of reinforced concrete frame structures. Some of these deficiencies are listed below:

(a) Structures with "strong beams" and "weak columns" exhibit little ductility and energy absorption.

(b) Low amounts of shear reinforcement, large stirrup spacings, high percentages of flexural steel, inadequate development of flexural reinforcement and inadequate anchorage of stirrup reinforcement have resulted in brittle failures and small amounts of energy absorption in beams.

(c) Low amounts of transverse reinforcement, large tie spacings, inadequate amounts of confinement reinforcement, large percentages of longitudinal steel and inadequate anchorage of both the longitudinal steel and the ties have resulted in brittle failures in columns.

(d) Small amounts of transverse reinforcement or lack of this reinforcement has resulted in brittle shear failures in the joints of frame structures.

(e) "Soft-storeys" which results from a reduction in the strength and the stiffness of a particular floor level have caused a number of dramatic examples of severe damage and complete collapse of reinforced concrete frame structures.

(f) "Short columns" resulting from the use of deep spandrel beams or partially infilled masonry walls between columns have clearly demonstrated that brittle shear failures occur in the columns.

(g) The practice of significantly reducing the size of columns in upper storeys of structures has led to many collapses of the upper storeys in concrete frame structures.

(h) There have been many examples of brittle punching shear failures in flat plate reinforced concrete frame structures.

(i) Many reinforced concrete frame structures (particularly flat plate frames) are too flexible and hence undergo large displacements causing significant P-delta effects which led to collapse and severe pounding between adjacent buildings.

(j) Torsional eccentricities due to eccentrically located walls or unsymmetrical plan configurations have caused severe damage and collapse.

It is noted that it is extremely difficult to significantly improve the ductility of all the beams, the columns and the joints in a non-ductile reinforced concrete frame structure. However, if the deficiency occurs only in certain members or only in small regions of the structure, it may be possible to apply special strengthening measures to the deficient regions.

Lack of adequate amounts of properly detailed ties which serve as both confinement reinforcement and shear reinforcement has been a major problem in past earthquakes. The flexural capacity may have to be increased to achieve a "strong-column", "weak-beam" structural system. Some common techniques for strengthening columns are:

(a) Add column ties and cover with concrete mortar.

(b) Cover the column with welded wire fabric and concrete mortar.

(c) Encase the column in a steel section (circular or rectangular) and grout the cavity.

(d) Cover the column with horizontal steel plates welded to steel angles at each corner of the column and grout as necessary. Fig. 2.4 illustrates this technique of repair being used on the columns of the 12 000 square metre, one storey Central Market building in Mexico City. This strengthening technique can easily be applied to the columns while the structure is still in use. The longitudinal corner angles significantly increase the axial load and flexural capacities and the horizontal plates increase the confinement and the shear resistance of the column.

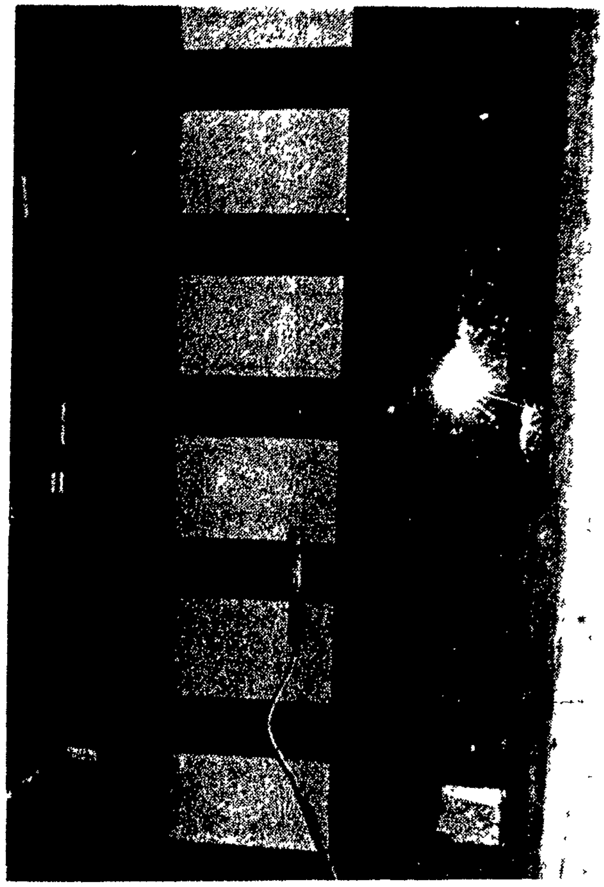


Figure 2.4 Strengthening of a Reinforced Concrete Column Using Steel Angles and Plates.

If the strengthening of the column significantly increases the stiffness of the member, then a separate analysis of the structure for lateral loading may be necessary to account for the change in stiffness.

Beams with inadequate amounts of shear reinforcement and excessive stirrup spacings can be locally strengthened using external stirrups. Figure 2.5 shows the use of additional exterior stirrups which are clamped to the existing beams of a reinforced concrete frame structure. This solution was suitable for the repair and strengthening of the ends of the beams in a 16 storey reinforced concrete frame structure in Mexico City having large reinforced concrete columns. As can be seen in Fig. 2.6 the ends of the

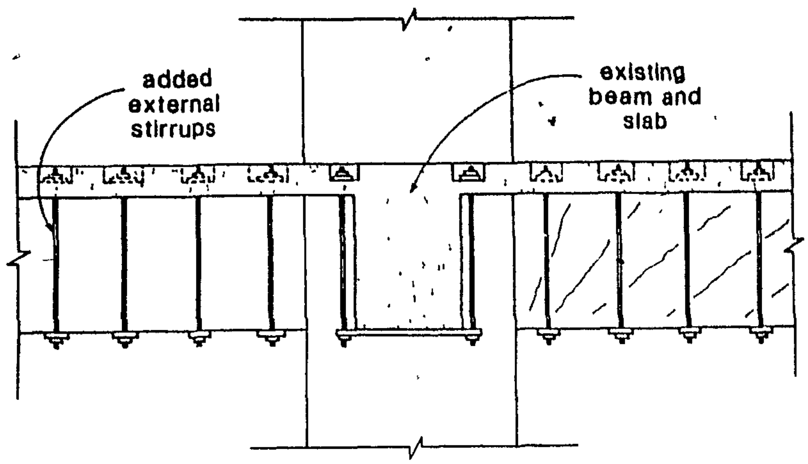


Figure 2.5 Cross-Section of Beam-Column Connection Showing the Use of External Stirrups.

Figure 2.6 Strengthening of Beam Using External Stirrups.

beams had suffered from significant diagonal tension cracks. The additional external stirrups play an important role in increasing the shear strength and the overall ductility of the beams but do not change the stiffness of the members.

In cases where additional structural elements such as walls will be added or in situations where a number of key structural components are strengthened, a more detailed analysis is necessary. In assessing the resulting structural system, it is necessary to account for the existing internal stresses in members before the upgrading and to account for redistribution of stresses that may take place after the strengthening. One of the goals should be to provide either a uniform or a smooth transition of stiffness and strength over the height of the structure and to remove torsional eccentricities wherever possible. One method of upgrading that has been successfully used for reinforced concrete frame structures is the strengthening and stiffening of some columns and beams. This upgrading technique is usually applied to the exterior frames since they are more easily accessible than interior frames. One approach involves the enlargement of column and girder sizes as well as the addition of longitudinal and transverse reinforcement to form cages around the existing columns and beams. Figure 2.7 illustrates this repair technique. The strengths and stiffnesses of both the beams and the columns are increased and closely spaced ties in the columns close to the existing joint region provide transverse reinforcement for the new enlarged joint region.

Existing shear walls can be strengthened by providing additional reinforcing steel on the outside of the walls and increasing the wall thickness with additional concrete[17]. This additional thickness can be obtained by shotcreting or guniting concrete to the roughened surface of the existing walls. This technique has the advantage of reducing the need for formwork but good workmanship must be assured in order to provide a uniform thickness of the wall. Shear connectors are usually required to ensure proper connection of the new and existing concretes. Care must be taken to anchor the ends of the horizontal and vertical reinforcement into adjacent columns and beams in order to provide an integral wall unit.

The addition of reinforced concrete shear walls just inside the exterior of the structure provides an efficient solution. It keeps the exterior architectural finish intact and

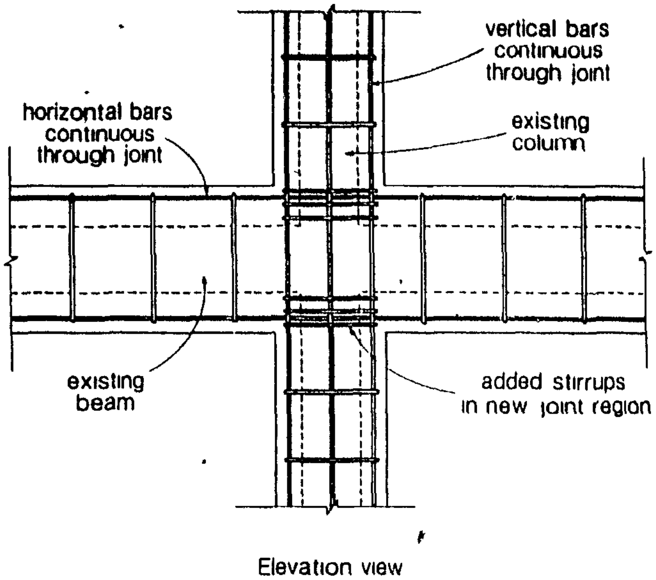


Figure 2.7 Method of Upgrading a Reinforced Concrete Frame Structure.

minimizes the modifications to the existing structure. The vertical wall reinforcement passes directly through the slab in order to ensure vertical continuity and to engage the slab in diaphragm action. The new shear wall requires the addition of footings which need to be accommodated around the existing column footings. Since the walls are significantly stiffer than the existing frame, they will attract most of the lateral load. If the new walls are designed and detailed to provide significant ductility (see Fig. 2.8), then the overall ductility of the structure can be significantly improved.

Additional lateral load resisting elements may be conveniently incorporated into extensions to existing facilities. An example of this upgrading strategy is the use of external steel braced frames which served as "buttresses" at the ends of the four wings of an existing hospital[12]. In this strengthening, a number of 30 m long steel plates, bolted to the existing floor slabs, served as collectors, reinforced the floor diaphragms and connected the existing structure to the added steel braced frames.

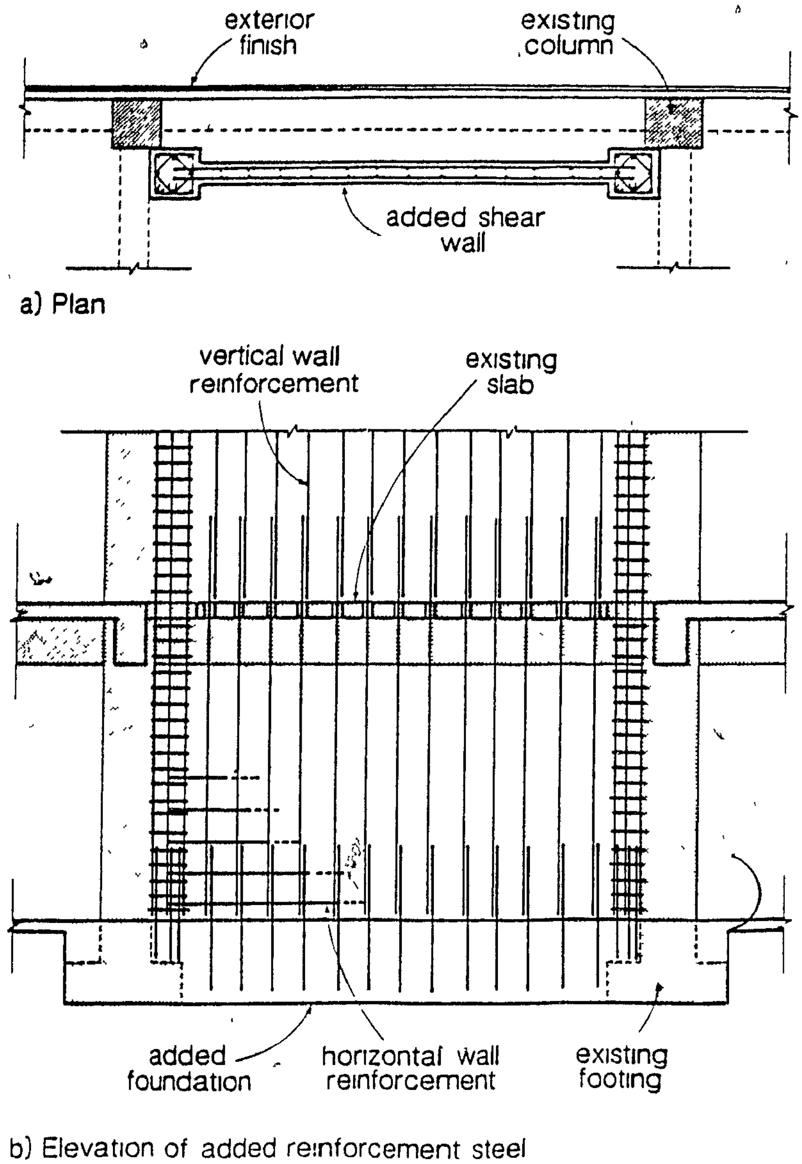


Figure 2.8 Addition of Reinforced Concrete Shear Walls to Existing Reinforced Concrete Frame Structure.

One method of improving the seismic performance of reinforced concrete structures is the addition of damping and energy absorbing devices. This type of retrofitting holds a great deal of promise for seismic upgrading. One application of this method was reported by Romero[23] on a 12 storey building in Mexico City after the 1985 earthquake. The damping was introduced by adding steel K-bracing equipped with an elastomeric damper located at the connection of the bracing to the floor beams. It was estimated that these damping devices increased the structural damping to about 20 percent of critical damping. Other innovative damping devices which can be used for the seismic upgrading of structures have been developed. A particularly promising method developed by Pall and Marsh[29] involves the use of friction damping devices which are installed in specially designed cross-braces. These friction dampers are capable of absorbing large amounts of energy, which is dissipated mechanically through friction.

2.4 Precast Concrete Structures

Common deficiencies in precast concrete structures arise because of the very nature of the construction, since the structures are composed of individual precast members which may lack proper connections between these elements. The lack of continuity in the structure together with the lack of stiff lateral load resisting elements makes many older precast structures susceptible to earthquake damage. One important deficiency is the lack of proper connections between floor and roof elements to enable the necessary diaphragm action to develop. In addition, the diaphragms may not be well connected to the vertical lateral load resisting elements. Deficiencies of the main lateral load resisting systems which are composed of precast wall panels are lack of connection between panels and lack of proper connections to the foundations. Many older precast structures were designed for seismic force levels which were too low due to the lower design force levels in older codes and due to the lack of recognition of the need to undergo proper design and detailing to achieve even modest levels of ductility.

In order to correct problems with lack of continuity of diaphragms, a reinforced cast-in-place topping can serve to connect the elements together. An alternative way of tying elements together is to apply exterior post-tensioned tendons in order to clamp the elements together.

In many situations, it may be necessary to add stiff lateral load resisting elements such as reinforced concrete shear walls. Added shear walls will probably require additional foundations and special details are required to connect the added shear to the existing precast concrete elements.

2.5 Steel Frames

Older steel frame buildings may have deficiencies in the connections and may have bracing systems, such as "tension only" bracing, which are known to have poor performance in severe earthquakes. Another common deficiency of some moment-resisting steel frames is the excessive drifts exhibited under lateral loads. For steel frames, the seismic force resistance level can be increased using relatively simple procedures which maintain the original system. Joints may be upgraded by replacing existing fasteners, such as bolts or rivets, with higher-strength fasteners. Where conditions permit, holes may be reamed to allow installation of larger diameter fasteners. Connections may be welded to achieve an increased load transfer capacity. Total replacement of a connection may frequently be the most expedient and economical method of improving a deficient joint.

The manner in which a member is loaded under service conditions may dictate the means for increasing its capacity[13]. In the case of compression members, methods such as reducing the unsupported length, increasing the cross-sectional area, or replacing sections with higher strength material may be used. Tension members, on the other hand, are usually strengthened by providing additional cross-sectional area, or by replacing them with the same size sections of higher-strength material.

The sequence of welding should be established so as to minimize warping and residual stresses. In situations where new material is added, it is necessary to account for the stresses in the existing members before the alteration and to carefully examine the load history of the member being upgraded. If the stiffness of a member is significantly altered, then a new analysis of the structure may be necessary in order to account for this change in stiffness.

In some situations, it may be necessary to add stiff lateral load resisting elements which can be used to stiffen the structure and to reduce any torsional eccentricity that may be present. Reinforced concrete shear walls have been added to steel frame buildings in order to increase the stiffness. Stud shear connectors welded to the existing steel frame help to transfer shear between the concrete and the steel. Vertical reinforcement in the shear wall can be made continuous over the height of the structure by passing reinforcing bars through holes drilled in the existing slabs. Steel cross-bracing in the form of X- or K-bracing or steel plate shear walls can also be used. Knee-braces, which are less obstructive than other forms of bracing, can also be added in order to increase the lateral stiffness of the structure. If knee-braces are added, columns should be checked for combined axial and bending stress and beams should be checked for knee-brace induced forces, and if necessary strengthened.

2.6 Foundations

The influence of the method of strengthening on the foundation system is an important consideration. Any alterations to the existing foundation system should be minimized since these are usually very costly[20]. But in most situations the strengthening and stiffening of the structure will induce an increase in load transfer to the foundations.

If the size of spread footings is shown to be insufficient by analysis, underpinning may be necessary to increase their size. If footings are still inadequate, they may be removed and replaced with larger footings. Where space permits, new footings may be placed on each side of an inadequate existing footing connected by transfer beams in order to distribute the load between the new and existing footings. Similarly, where space permits caissons or piles may be used in lieu of spread footings and placed on each side of the existing footing.

Pile footings are usually more difficult to strengthen than spread footings. Space for adding new piles must be available, including vertical clearance for the pile driver if driven piles are used or space for the drilling rig where drilled and cast-in-place concrete piles are used. Where calculations show that multiple pile footings do not meet the required capacity, it may be necessary to remove the existing pile cap, place additional piles, and provide a new pile cap.

In situations of soft soil conditions and where vertical clearance is not available, segmental piles can be used. These piles, which have been successfully used in Mexico City, are hydraulically pushed in short segments through holes created in the existing foundation by jacking against a reaction frame attached to the structure. After hydraulically pushing the "lead-pile" segment, a prestressing tendon, anchored into the "lead-pile"segment, is threaded through a central hole in the next segment, which is then pushed. The process is repeated until the pile has reached the required depth.

The completed pile is then post-tensioned and joined to the existing structure by a reinforced concrete pile cap. Another type of segmental pile used in Mexico City utilizes welded connections. Each pile segment has steel plates attached to the its ends which are used to connect the subsequent segments together by welding of these plates around their exposed edges. Circular pile cross-sections are most suitable since they can be easily manoeuvred into position by rolling the segments along planks. The applications of both of these types of foundation repair techniques are illustrated in Chapter 3.

Some methods of soil compaction and stabilization such as pressure grouting or intrusion grouting with cement grout or chemical grouting can be used to increase the bearing capacity of the soil. These techniques require a thorough investigation of the underlying soils since the selection of the appropriate technique is strongly related to the specific soil characteristics.

CHAPTER 3

SEISMIC UPGRADING IN MEXICO CITY

3.1 The 1985 Mexican Earthquake

The September 19, 1985 magnitude 8.1 earthquake and its magnitude 7.5 aftershock caused unprecedented damage to structures in Mexico City. It is recognized that the nature of the ground motion together with the modification of this motion by the soft soil conditions in the lake zone resulted in severe strong motion characteristics[1]. The subsoil amplified the horizontal acceleration by a factor of about five giving a maximum acceleration of about 0.20g at one accelerograph site. A key feature of the motion was a period of about 2 s and a long duration of strong ground motion. There were 5 cycles of motion at about 0.20g with horizontal accelerations of about 0.10g over a duration of 22 s. One month after the earthquake, the emergency code regulations[11,18] appeared which drastically changed many of the provisions of the 1977 building code[10]. A summary of some of these changes[24,25] for concrete structures in soft soil region (Zone III, see Fig. 3.1) is given below:

(1) The seismic lateral force coefficient for design in Zone III was increased from 0.24g to 0.40g, an increase of 66%. In zone II, this coefficient was increased from 0.20g to 0.27g.

(2) The importance factor was increased from 1.3 to 1.5.

(3) the reduction of lateral force due to ductility was significantly reduced in some cases.

(4) The resistance factors for shear, torsion and for axial load plus bending were reduced.

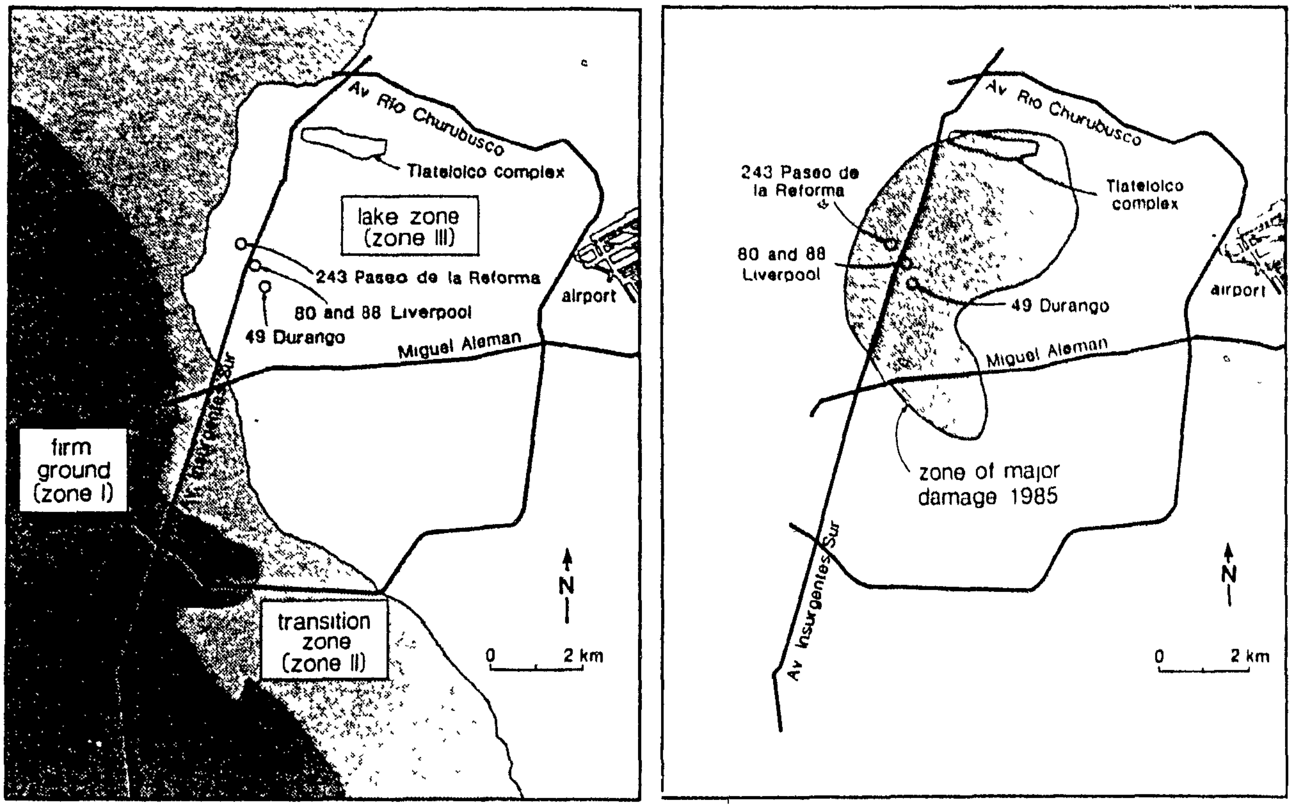
(5) Office live loads were increased from 1.47 kPa to 1.77 kPa.

(6) Column reinforcement details were improved to increase confinement and to prevent the buckling of longitudinal reinforcing bars.

(7) The minimum separation requirements were emphasized.

These emergency code changes applied to all important structures in all zones, to all structures in Zone III that were either damaged or under construction on September 19, 1985 and to all future construction. For undamaged structures in Zone I and II which were under construction at the time of the earthquake, only the minimum separation requirements applied. In addition, a height restriction of 4 storeys was placed on new construction in Zone III. These changes gave rise to new lateral load force levels which were significantly increased for Zone III (more than doubled for some types of structures).

A number of case studies of structures which were repaired and upgraded following the 1985 Mexican earthquake are described below. These structures are all located in seismic Zone III (lake zone), that is, in the region of severe damage, as shown in Fig. 3.1. It is noted that the significant increase in design force levels resulted in extensive repair and strengthening measures in order to meet the severe changes to the building code. It is important to view the following examples in the correct context, that is, that these strengthening measures took place in Mexico City after the 1985 earthquake. Therefore, these examples provide qualitative rather than quantitative guidance for the structural upgrading of existing structures in Canada.



a) Sub-soil zones

b) Zone of major damage in the 1985 earthquake

Figure 3.1 Location of Structures Used in Case Studies, Seismic Zones[10] and Zones of Major Damage in the 1985 Mexican Earthquake[18].

3.2 Case Study 1: Eleven Storey Office Building at 243 Paseo de la Reforma

The eleven storey office building situated at 243 Paseo de la Reforma in Mexico City is located in subsoil zone III (lake zone) in the region of severe damage from the 1985 Mexican earthquake (see Fig. 3.1). The 2 bay by 7 bay structure shown in Fig. 3.2 consisted of reinforced concrete columns and waffle slabs. As can be seen, a floating foundation with inverted barrel shells was used to overcome the difficult soil conditions (i.e., highly compressible clay with an extremely high water content).

Although no major damage was apparent, the foundation and the structure had to be upgraded to comform to the emergency code provisions. A total of 56 precast concrete segmental piles were added in order to increase the foundation capacity (see

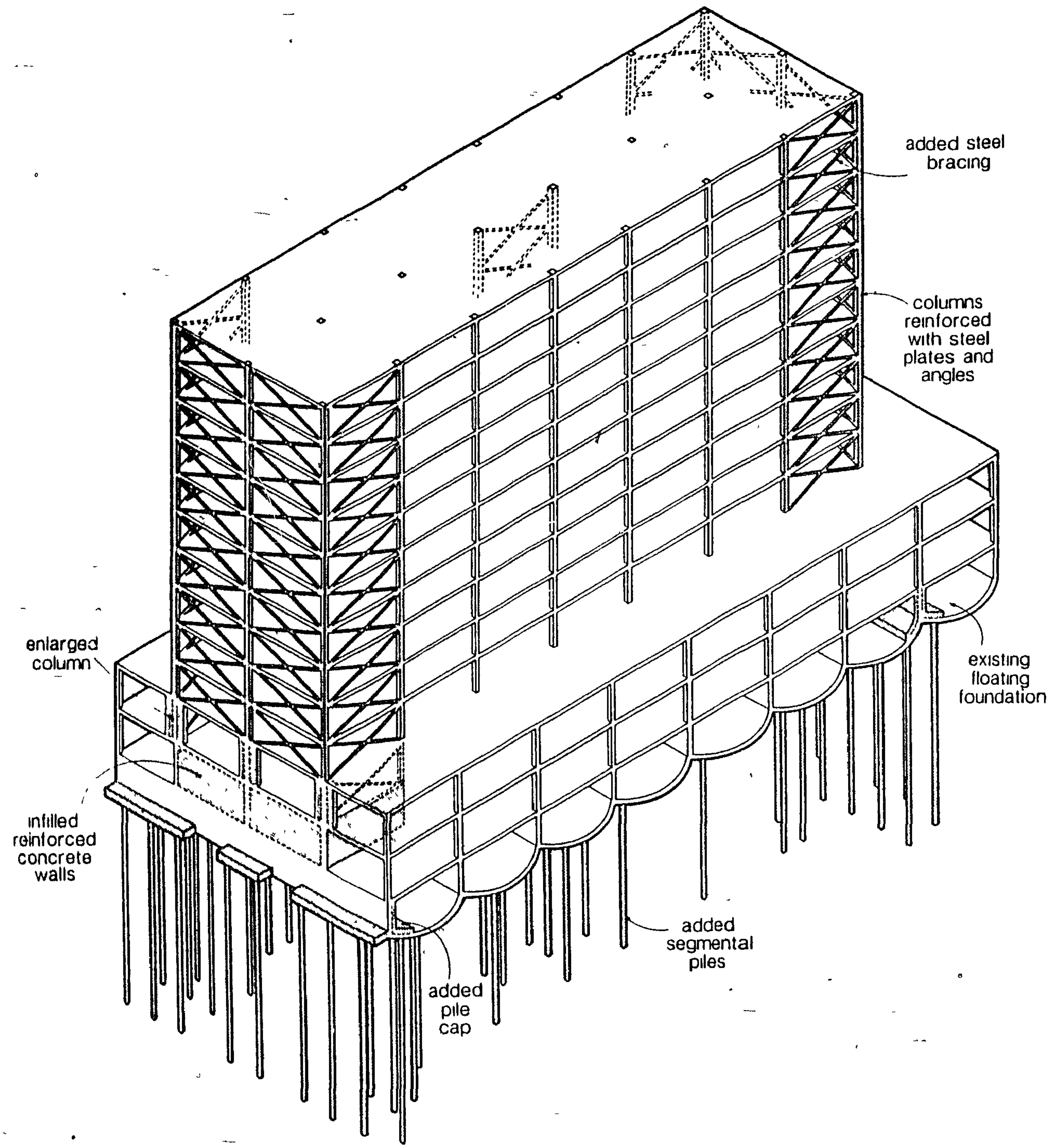


Figure 3.2 Strengthening of Eleven-Storey Office Building at 243 Paseo de la Reforma in Mexico City.

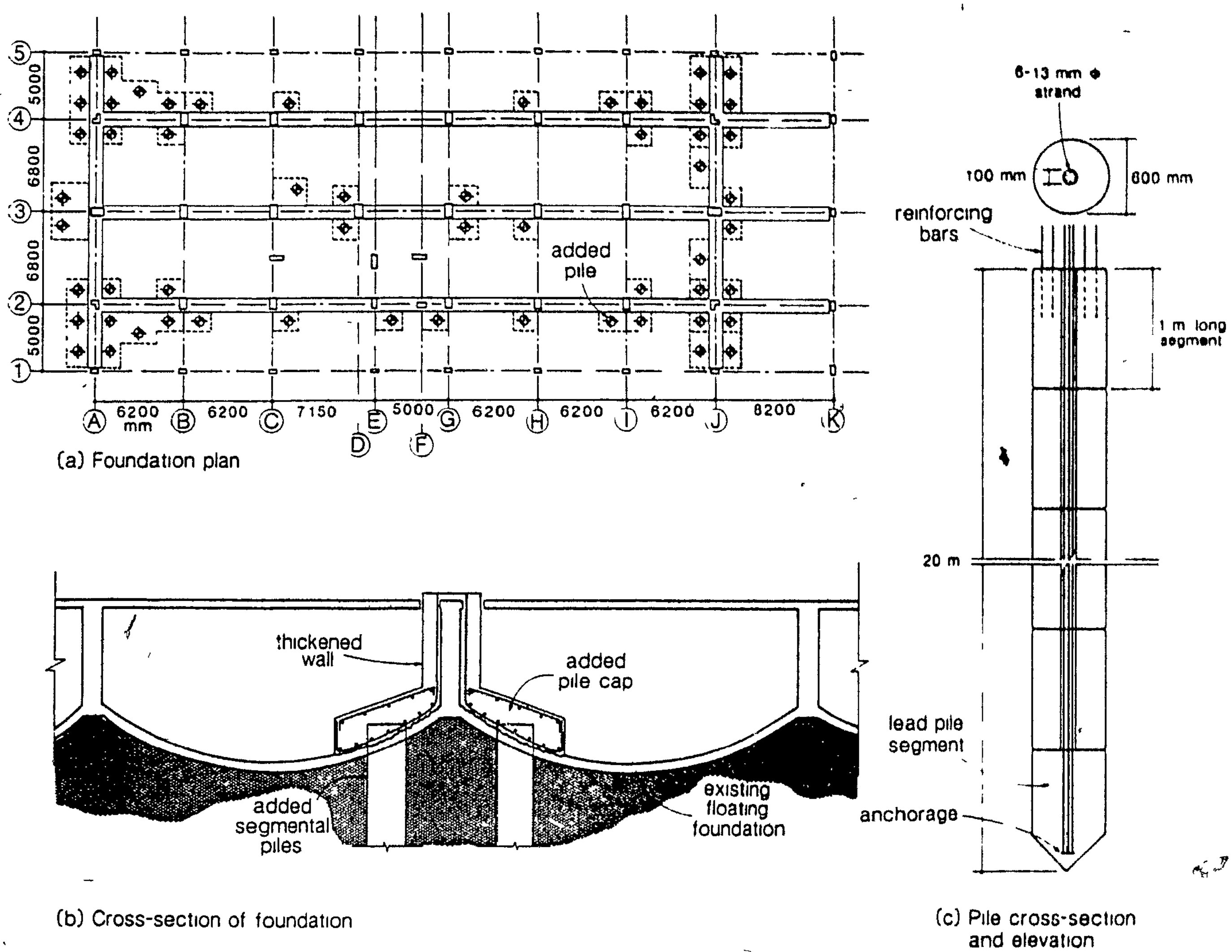


Figure 3.3 Foundation Strengthening Using Precast Concrete Segmental Piles.

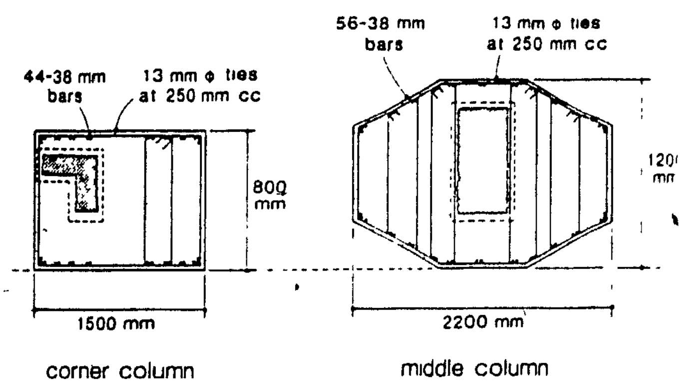
Fig. 3.3). The 20 m long end-bearing piles consisted of 20 - 1 m long segments. This length of piles was required to reach the stiffer sand layer located at about 20 m below the foundation level.

As can be seen from Fig. 3.2 the top ten storeys were strengthened and stiffened by steel cross-braces located at both ends of the building and in an interior bay. The use of steel cross-braces in the configuration shown provides an efficient means of strengthening and stiffening the structure for lateral loads (especially for torsional effects). The cross-bracing also permits flexibility in the use of the interior space and minimizes the obstruction to natural lighting. In order to provide access to the main street, cross-bracing was not permitted in the end bent of the ground floor level. It was therefore necessary for the engineer to seek an alternative method of strengthening the ground floor level in an attempt to avoid creating a "soft-storey". Hence the existing columns were significantly enlarged at this level (see Figs. 3.2 and 3.4). At the basement level, a shear wall was added to transmit the loads to the foundation. Figure 3.5 shows the details of the steel cross-braces used to reinforce the structure. Structural steel angles were placed at the corners of the columns and horizontal steel plates were welded to these corner angles. The tension-compression braces consisted of two 203 mm deep channels welded together by 10 mm thick batten plates. The braces were welded to steel plates attached to the columns. In addition, the braces are welded to 13 mm thick steel cover plate on both the top and bottom surfaces of the slab. These steel plates were bolted through the thickness of the slab as shown in Fig. 3.5b, with 13 mm diameter bolts. The purpose of the horizontal steel plates is to provide shear connection between the cross-bracing and the slab diaphragm, to assist in the transmission of column tensions through the joint region and to increase the punching shear resistance of the slab.

Figure 3.4 illustrates the method used to strengthen the ground floor columns in the end bay. The first step was to chip the column faces in order to expose the existing reinforcement and roughen the concrete surfaces. As can be seen, the column sizes have been significantly increased and large amounts of steel reinforcement have been added. A beam was added beneath the floor slab in order to connect the tops of the ground floor columns (see Fig. 3.4a). The steel corner angles were extended from the second floor columns some distance into the ground floor columns in order to ensure continuity of the vertical reinforcement.

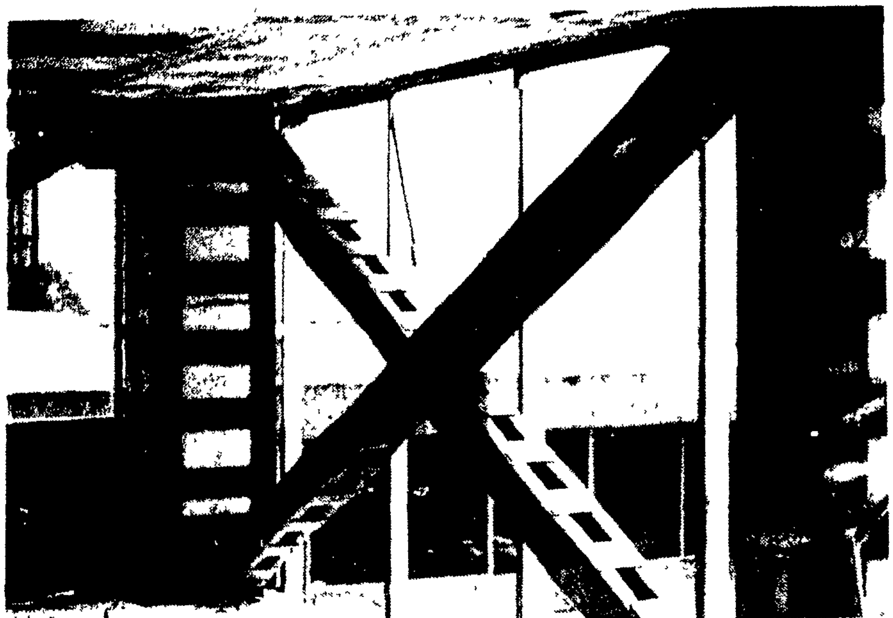
For the interior braced bay, reinforced concrete shear walls were used instead of steel cross-bracing at the ground floor and basement levels. Figure 3.6a illustrates the

a) Overall view

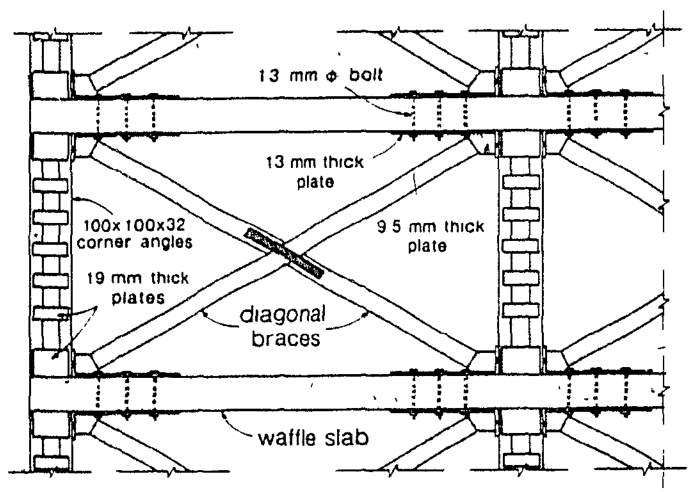


b) Cross-section of ground floor columns

Figure 3.4 Transition from Steel Cross-Bracing to Enlarged Columns at Ground Floor Level.



a) Interior view



b) Structural details

Figure 3.5 Tension-Compression Steel Cross-Bracing.

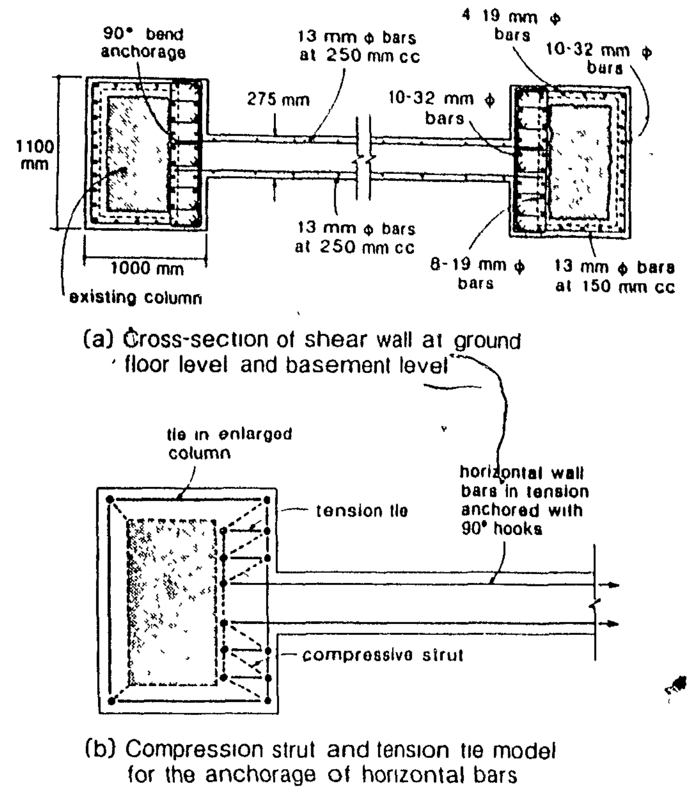


Figure 3.6 Cross-Section of Shear Wall at Ground Floor Level and Basement Level.

cross-sectional details of this added shear wall. As can be seen, the column sizes were increased to form the ends of the shear wall. The enlargement of the column and its reinforcement provided some degree of confinement for anchoring the horizontal wall reinforcement. The compression strut and tension tie model shown in Fig. 3.6b is useful for the design of the additional ties required in the enlarged column. The yield force of the horizontal wall bars is transferred to the existing column by compressive struts and tension ties. The model illustrates the need for tension ties in the form of smaller column ties on the inner faces of the enlarged column. The forces in these smaller column ties can be found from statics and the tie size and spacing can be chosen.

The cross-braces can be seen from the inside of the building and offer the occupants a sense of security (see Fig. 3.7). From the outside of the building the cross-braces are not visible due to the use of a reflective glass curtain-wall (see Fig. 3.8).

Figure 3.7 Interior View of Nearly Completed Eleven-Storey Building at 243 Paseo de la Reforma in Mexico City.

3.3 Case Study 2: Nine-Storey Office Building at 88 Liverpool

The nine-storey office building located at 88 Liverpool in Mexico City was constructed in 1971. It is located in subsoil zone III (see Fig. 3.1) and was severely damaged during the 1985 Mexican earthquake. The structure had a period of about 1.3 s, which was close to the 2 s period of the ground motion. This together with the large accelerations experienced during the earthquake resulted in severe damage. Figure 3.9 shows the structure in its damaged condition with temporary timber shoring while the building was being repaired. A number of columns failed in the structure. Figure 3.10 illustrates the shear failure of a fourth floor exterior column. The extremely large tie spacing, which was inadequate to provide confinement of the concrete core, together with the poor anchorage details of the ties (i.e., only 90 degree bend anchorages at the ends of the ties) were the primary causes of the shear failure. As can be seen, after spalling of the concrete cover, the column ties lost their anchorage and the longitudinal bars buckled.

Figure 3.11 shows the failures of the joint region of the corner column. In ad-

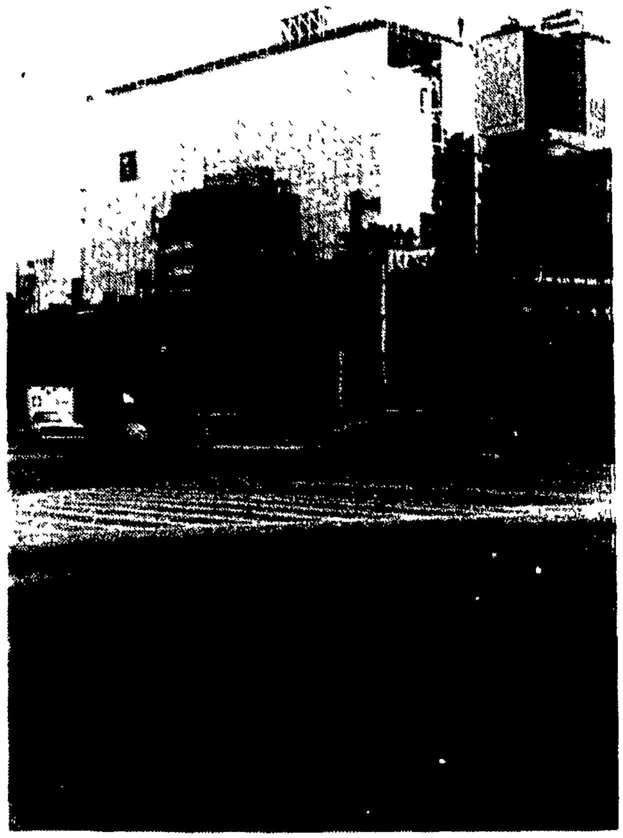


Figure 3.8 Exterior View of Nearly Completed Eleven-Storey Building at 243 Paseo de la Reforma in Mexico City.

dition to the damage to the columns, some beams displayed buckling of the bottom longitudinal bars due to excessive stirrup spacing (see Fig. 3.12).

A plan view of the structure is shown in Fig. 3.13. Before repair the structure consisted of two distinct parts separated by a 100 mm wide construction joint between the columns and the beams along column lines 5 and 6. The structural framing consisted of beams and columns with a 100 mm thick one-way slab spanning between secondary beams.

The structural strengthening measures included the following:

(a) The existing inverted barrel shell floating foundation was strengthened by hydrauli-

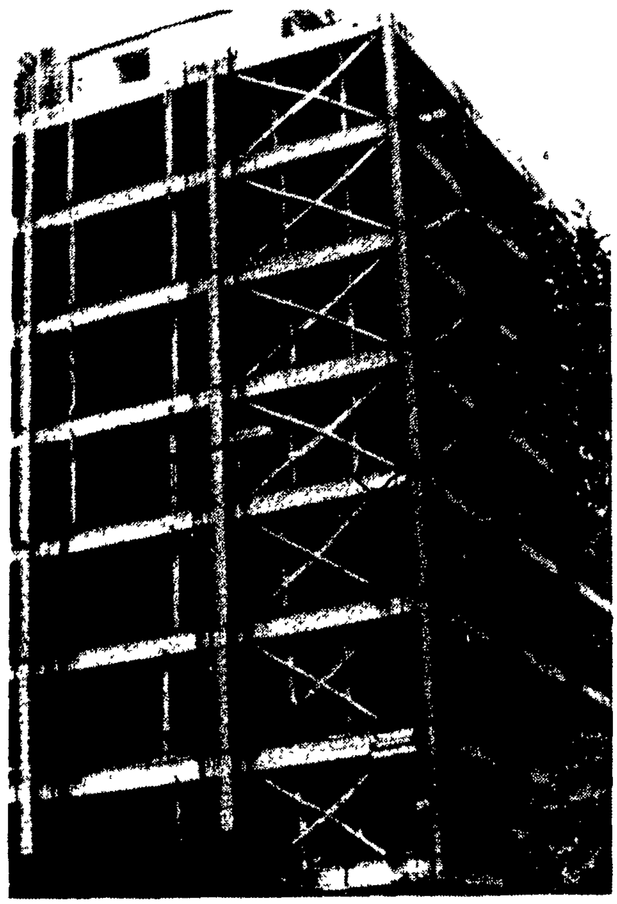


Figure 3.9 Overall View of Nine-Storey Building at 88 Liverpool During Strengthening.

cally pushing 39 precast concrete segmental piles similar to the details shown in Fig. 3.3. Figure 3.14 shows one of the 1 m long segment of the pile ready to be hydraulically pushed. Note the prestressing tendon passing through the centre hole.

(b) The two separate parts of the building were joined by eliminating the construction joint in order to increase the lateral load resistance and stiffness. This measure also removes the possibility of pounding between the adjacent parts. This was achieved by encasing the adjacent columns and beams along the construction joint in enlarged reinforced concrete columns and beams as shown in Figs. 3.15a and 3.15b. In addition, the beams along column lines B and C were enlarged over a portion of their length to

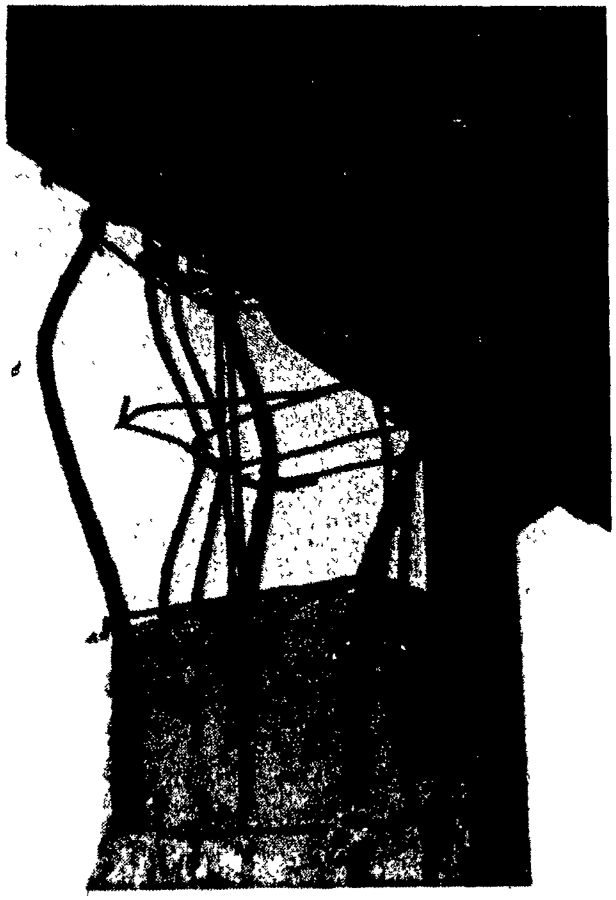


Figure 3.10 Shear Failure of a Fourth Floor Exterior Column[26].

aid in transferring the force into the slab diaphragm (see Fig. 3.15c).

(c) Figure 3.16 shows the shear walls and cross-braces which were added to three bays along column line 9. This reinforcement consisted of a five storey high reinforced concrete shear wall in the centre bay and reinforced concrete cross-braces elsewhere in the bent. The openings in the cross-braces were later filled with masonry, a common form of construction in Mexico. The reinforced concrete shear wall was placed between the existing columns, which were enlarged and reinforced in order to increase both the stiffness and strength of the wall and to provide a zone capable of anchoring the horizontal wall reinforcement (see Fig. 3.16b). The strut and tie model shown in Fig. 3.16c provides a useful tool for visualizing the flow of the forces and the design

Figure 3.11 Joint Failure of Corner Column.

of the reinforcement necessary to anchor the horizontal wall bars. An additional four reinforced concrete walls were added as shown in Fig. 3.13.

(d) The exterior columns and spandrel beams along column lines A and 1 were significantly increased in size and additional reinforcement was added. The reinforcement details for typical enlarged column at the ground level can be seen in Figs. 3.17 and 3.18. Note that the longitudinal reinforcement in the columns passes through the floor slabs. Figure 3.19 shows the reinforcement and formwork for the exterior columns and spandrel beams.

One of the most difficult parts of structures to upgrade are the joint regions between beams and columns or beams and slabs. Fig. 3.20 illustrates one method of

Figure 3.12 Buckling of Bottom Longitudinal Beam Bars[26].

strengthening a beam-column joint. The enlargement of both the column and the beam enables an increase in the amount of both transverse and vertical reinforcement passing through the joint. The resulting increase in joint reinforcement and size significantly improves the joint behaviour. This method was used to strengthen the beams, columns and joints in the structure at 88 Liverpool (see Fig. 3.19). It is noted that special attention was given to the end anchorage details for the column ties. All of the ties were anchored around longitudinal bars with bends which were greater than 135 degrees.

The combined effects of joining together the two parts of the structure, the enlargement of the columns together with the addition of reinforced concrete shear walls and braces resulted in a significant stiffening of the structure. This stiffening lowered the first period from about 1.3 s to about 0.7 s. This shift in the period of the structure away from the dominant 2.0 s natural period of the ground motion would lead to significant improvement in the response of the building. In the design of the strengthening a ductility factor, Q, of 2.0 was used. Figure 21 shows the structure near completion.

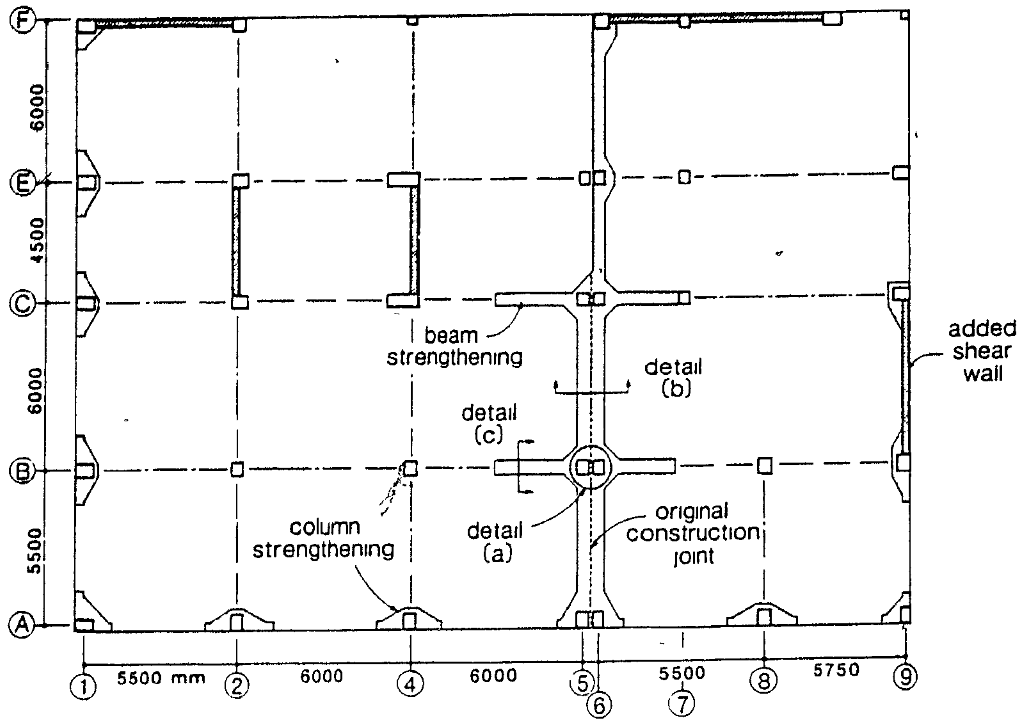


Figure 3.13 Plan View of Nine-Storey Structure at 88 Liverpool (see details in Fig. 3.15).

3.4 Case Study 3: Nine-storey Office Building at 80 Liverpool

The nine-storey office building situated at 80 Liverpool in Mexico City was originally constructed in 1971. This structure is located directly across the street from the structure described in Case Study 2 (see Fig. 3.21).

The structural framing consisted of moment-resisting frames combined with masonry infilled walls. A plan view of a typical floor is shown in Fig. 3.22. A total of 25 segmental, precast concrete piles, similar to those shown in Fig. 3.3 and Fig. 3.14, were added to the existing pile foundation. The reinforced concrete shear walls shown in Fig. 3.22 replaced the masonry infilled walls. As can be seen from Figs. 3.22 and 3.23 heavy columns and deep beams replaced the masonry infilled walls in the structure

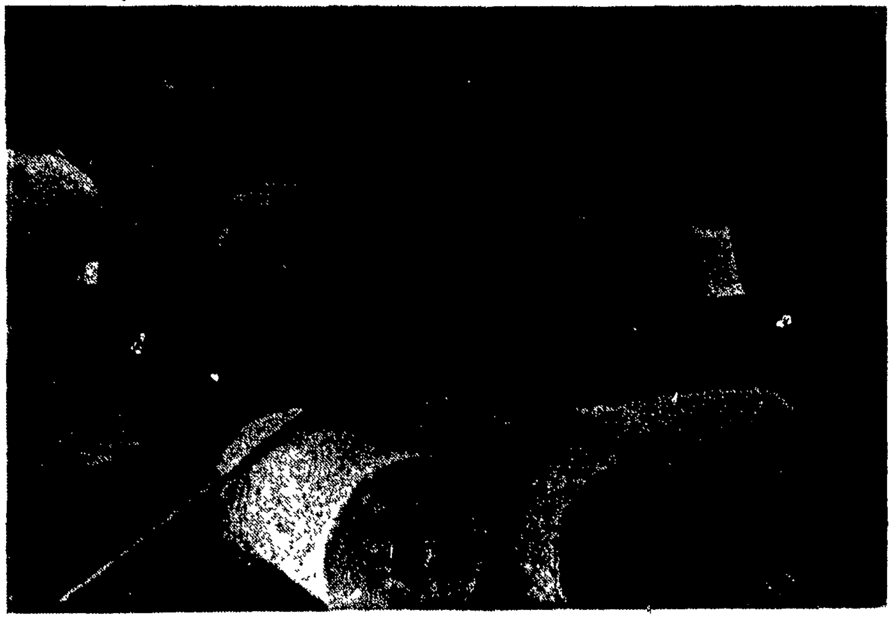


Figure 3.14 Precast Pile Segments and Reaction Frame Used for Hydraulic Pushing of Piles. (Photograph courtesy of Octavio Armengol).

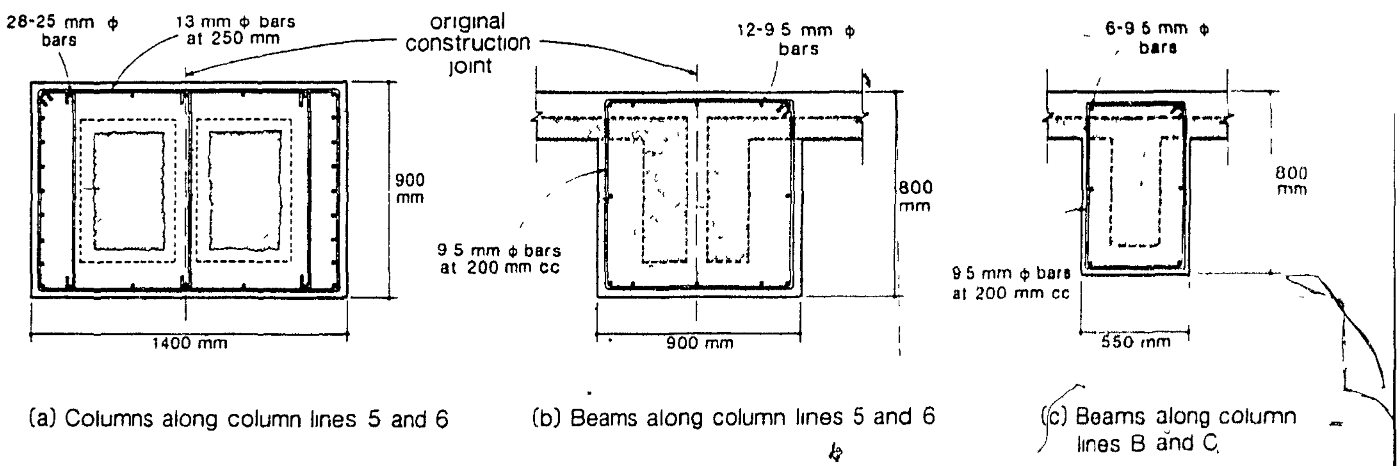
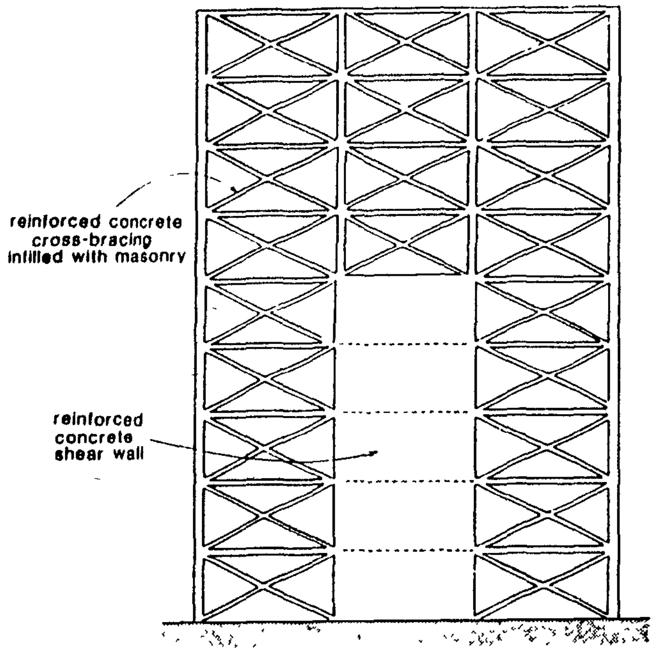
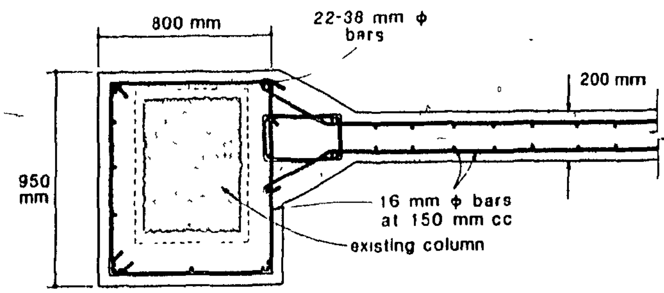


Figure 3.15 Cross-Sections of Members which were Strengthened to Eliminate Construction Joint.

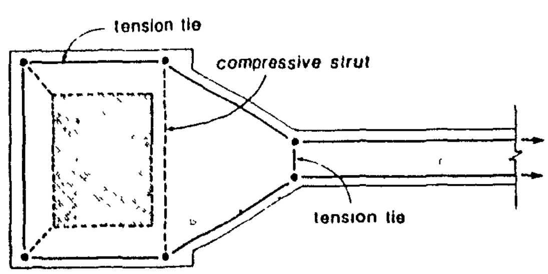
along column line F. Continuous steel cross-braces were added along column lines 1 and 3 (see Figs. 3.23 and 3.24). the tubular diagonal members were made from two welded channels. The cross-braces were connected, using welded steel brackets, to 13 mm thick steel plates which in turn were bolted to the frame at each floor level (see Figs. 3.24c and 3.24d). It is noted that the placement of this tension cross-bracing



(a) Elevation view

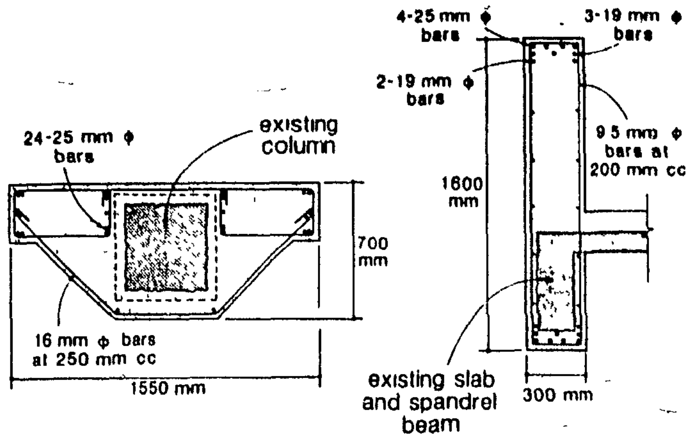


(b) Cross-section of shear wall



(c) Compression strut and tension tie model
for the anchorage of horizontal bar

Figure 3.16 Strengthening and Stiffening of Frame along Column Line 9.



(a) Cross-section of ground floor column

(b) Cross-section of spandrel beam at second floor level

Figure 3.17 Typical Enlarged Edge Column and Edge Beam along Column Lines A and 1.

on the exterior of the structural frame was easy to install and hence reduced the construction time considerably. Due to architectural constraints, two very different types of lateral load resisting elements were added; tension steel cross-bracing along column lines 1 and 3 and reinforced concrete shear walls along column line 2. In these situations, it is essential that every effort be made to ensure that differences in behaviour of these two systems do not result in increasing the torsional eccentricity. An additional design consideration is that the connections for the cross-bracing introduce eccentric loads on the beam-column connections.

Views of the finished structure are shown in Figs. 3.25 and 3.26. Due to the use of reflective glass the cross-braces are not visible from the exterior of the structure but are clearly visible from the interior. This application of cross-bracing demonstrates that if the bracing is carefully detailed it can become a positive architectural feature.

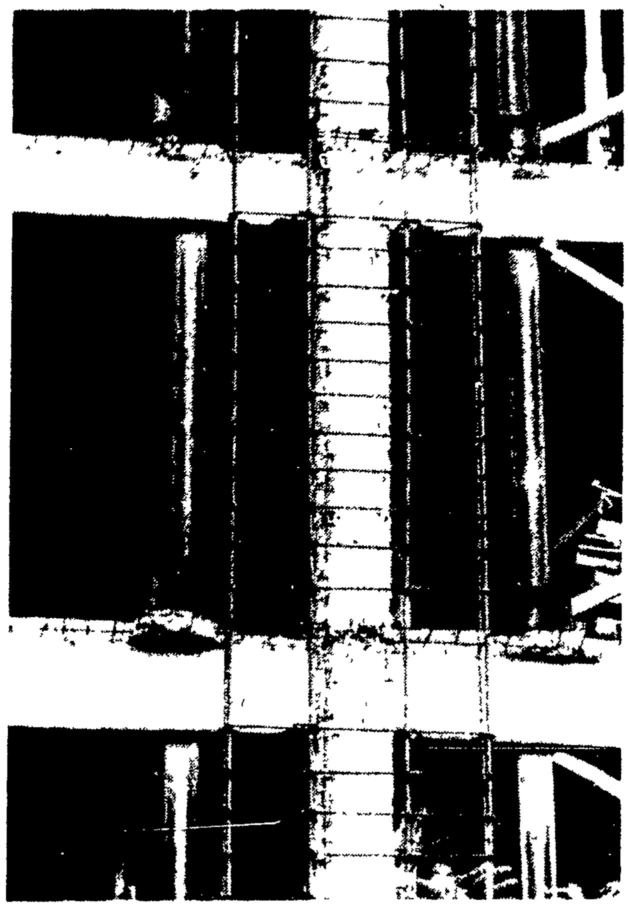


Figure 3.18 Typical Enlarged Edge Column.

3.5 Case Study 4: The Multi-Storey Tlatelolco Apartment Buildings

Figure 3.27 shows a plan view of the Tlatelolco multi-storey apartment complex which is located in the lake region close to the transition zone (see Fig. 3.1). The depth of soft soil varied from 24 m in the east to 34 m at the west end of the complex. There was a total of 102 apartment buildings housing 120,000 people. After the 1985 earthquake 8 structures had to be demolished, 60 had to have minor repairs and 34 buildings required structural repairs. The total cost of the repairs is estimated to be 38,500 million pesos (approximately $40,000,000 Canadian). The structures were built in the early 1960's and were inaugurated in 1964. The modified lateral force coefficient used in the original design was 0.06 g. According to the 1985 Emergency Code regu-

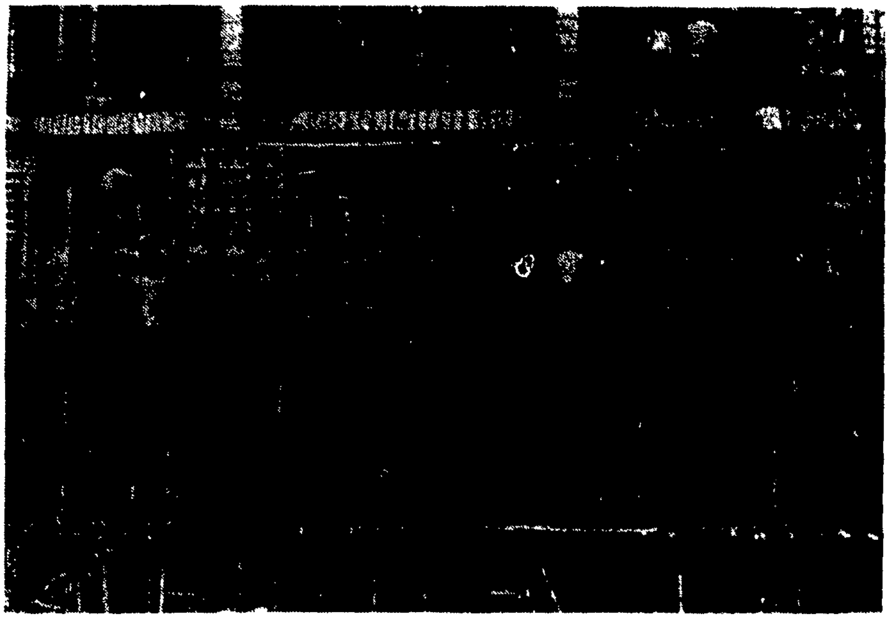


Figure 3.19 Reinforcement and Formwork of Enlarged Exterior Columns and Spandrel Beams. (Photograph courtesy of Octavio Armengol).

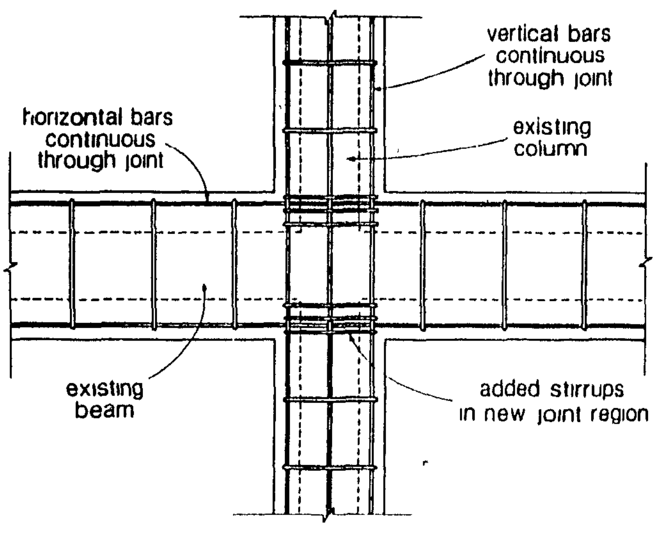


Figure 3.20 Method of Increasing Joint Shear Resistance.

lations, the elastic lateral force coefficient used in the design of the reconstruction was 0.40 g. For those structures strengthened with beams and columns, a ductility factor, Q, of 4 was used giving a modified lateral force coefficient of 0.10 g. For structures with stiff walls, Q was taken as 3, giving a modified lateral force coefficient of 0.133 g.

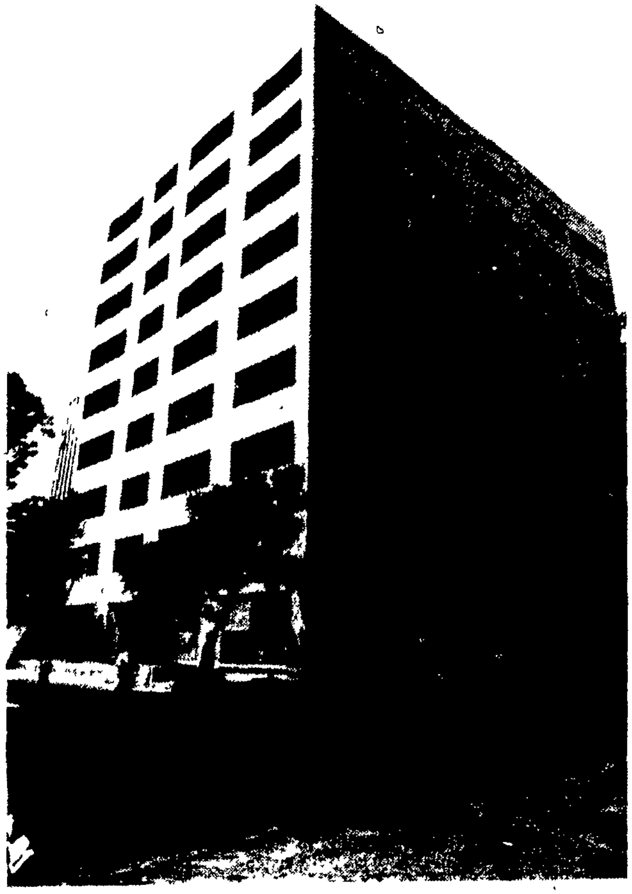


Figure 3.21 View of Nearly Completed Office Building at 88 Liverpool.

A major feature of the repair of the structures was the strengthening of the foundations. In general, the foundations were repaired by extending the existing foundation to increase the bearing area. These rigid box foundation extensions were carried out above the water level which is at a depth of about 3 m. As can be seen from Fig. 3.27, it was decided to both reduce the height as well as reinforce some of the structures.

Figure 3.28 shows a 14 storey building with offset floors which, due to this poor lateral load resisting system, suffered severe damage along the line of interconnection of the offset floors. In the short direction, the lateral load resisting elements consisted of reinforced concrete shear walls for the bottom 5 storeys and reinforced concrete diagonal braces, with infilled masonry for the upper storeys. The measures taken to

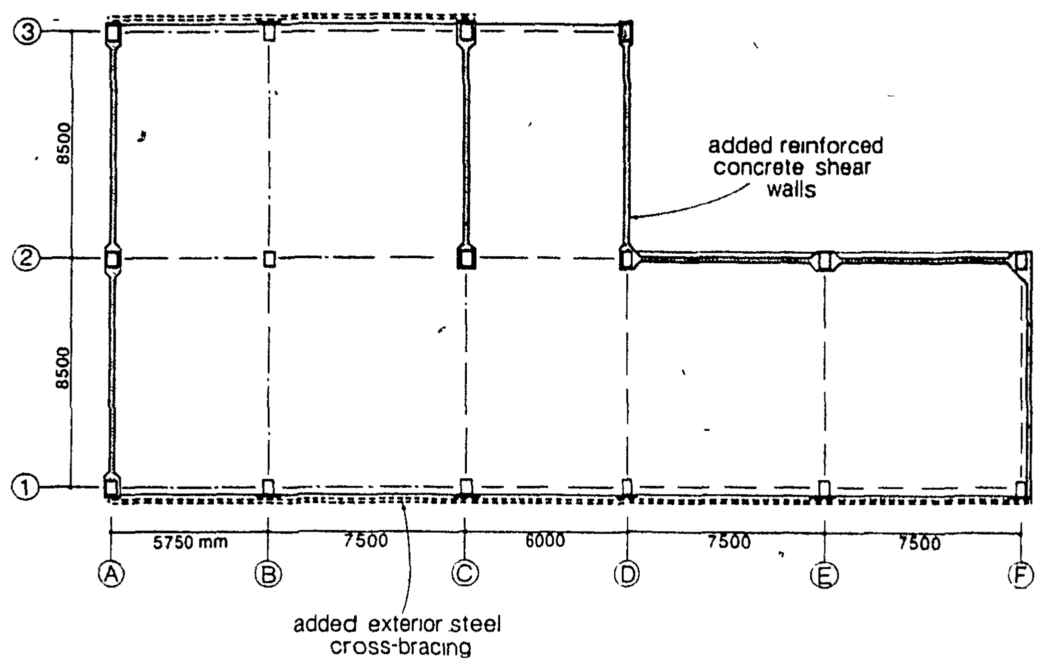


Figure 3.22 Plan View of Nine-Storey Structure Located at 80 Liverpool in Mexico City.

repair and strengthen this structure included the following:

(a) The structure was reduced to 7 storeys in height as shown in Fig. 3.29.

(b) Enlarged exterior reinforced concrete columns were added along the lines of interconnection of the offset floors.

(c) At the ends of the structure the diagonal braces were replaced by reinforced concrete shear walls. In addition, 2 wing walls, perpendicular to the shear walls, were also added at each end of the structure as shown in Fig. 3.30. These walls, which were channel-shaped in cross-section, were oriented such that the flanges of the channels pointed outwards, thus minimizing the obstruction of the interior space.

Figure 3.31 shows the L. Cardenas Building just after the 1985 earthquake. This structure which consisted of 3 parts separated by 2 construction joints suffered severe damage due to pounding of the adjacent parts. A similar structure, the Nuevo Leon

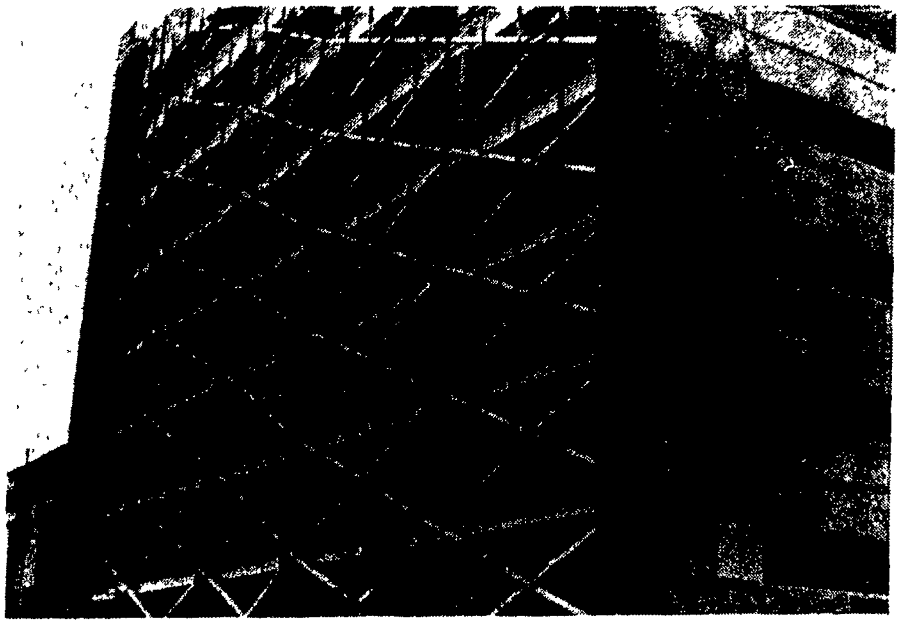


Figure 3.23 Strengthening of Nine-Storey Office Building Showing Continuous Cross-Bracing. (Photograph courtesy of Octavio Armengol).

building, suffered total collapse of 2 of the 3 parts, with the third part being severely damaged. The Nuevo Leon building was demolished and the remaining 8 similar buildings were strengthened by replacing the infilled masonry cross-braced walls by shear walls (see Fig. 3.32) and by adding beams and columns (see Figs. 3.33 and 3.34). Since the adjacent parts pounded against each other during the earthquake, the separations between the parts were increased to 700 mm.

Slip-forming was used to form reinforced concrete shear walls replacing reinforced concrete braces in some 21 storey apartment buildings.

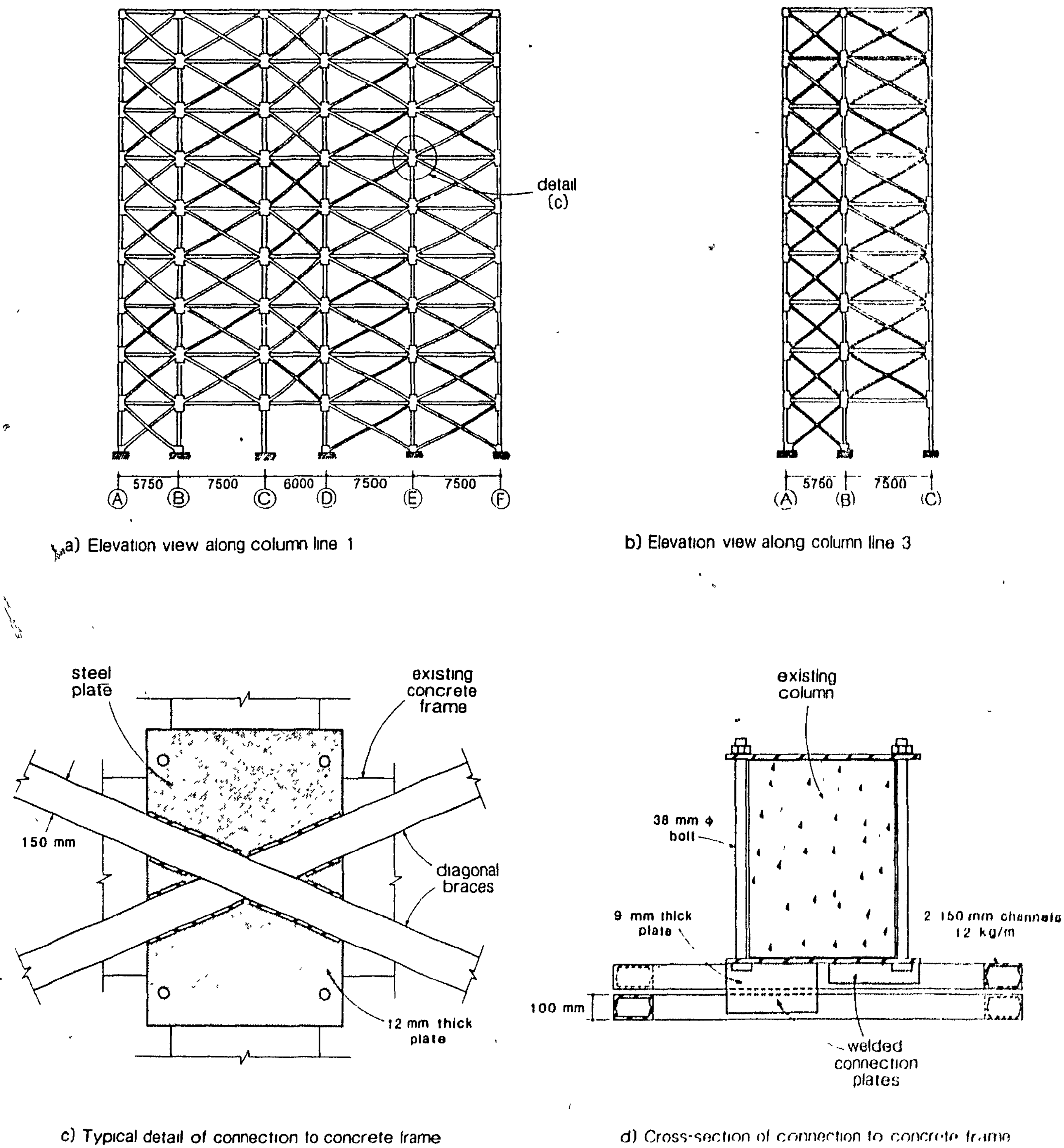


Figure 3.24 Added Cross-Bracing.

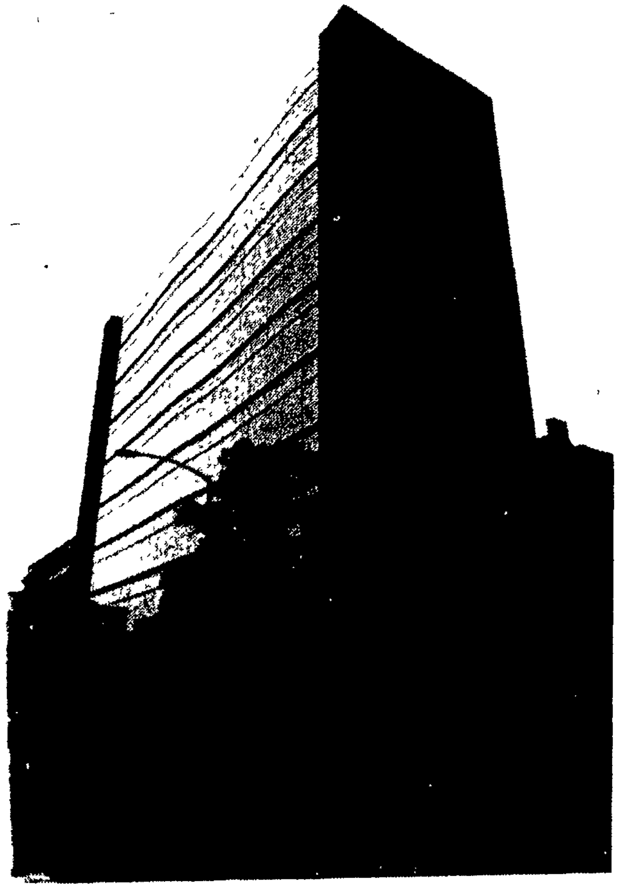


Figure 3.25 Exterior View of Office Building at 80 Liverpool.

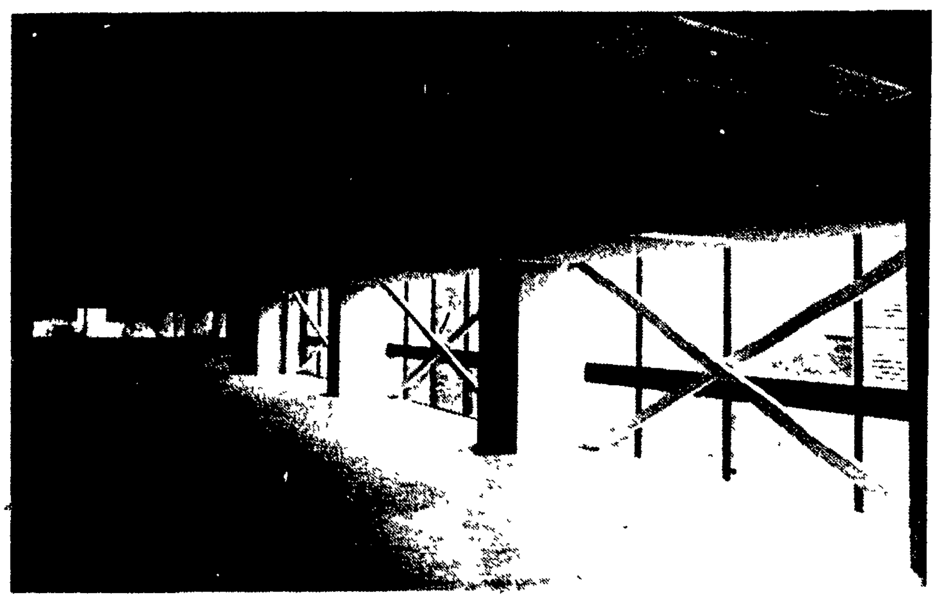


Figure 3.26 Interior View of Finished Structure Showing Diagonal Bracing.

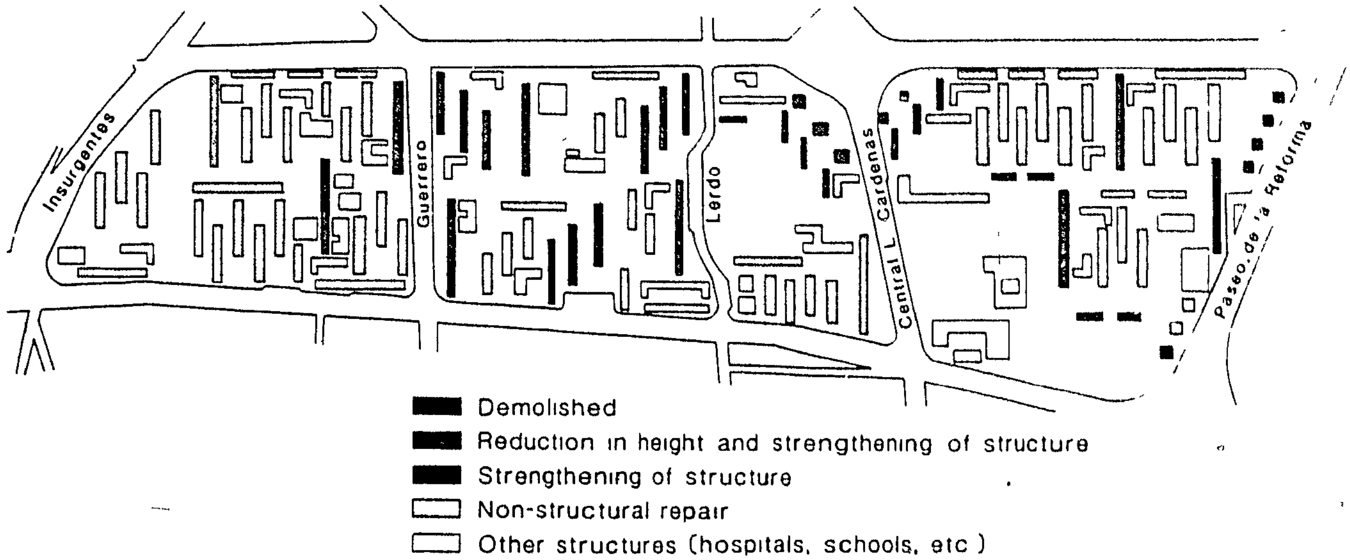


Figure 3.27 Plan View of Tlatelolco Multi-Storey Apartment Complex.

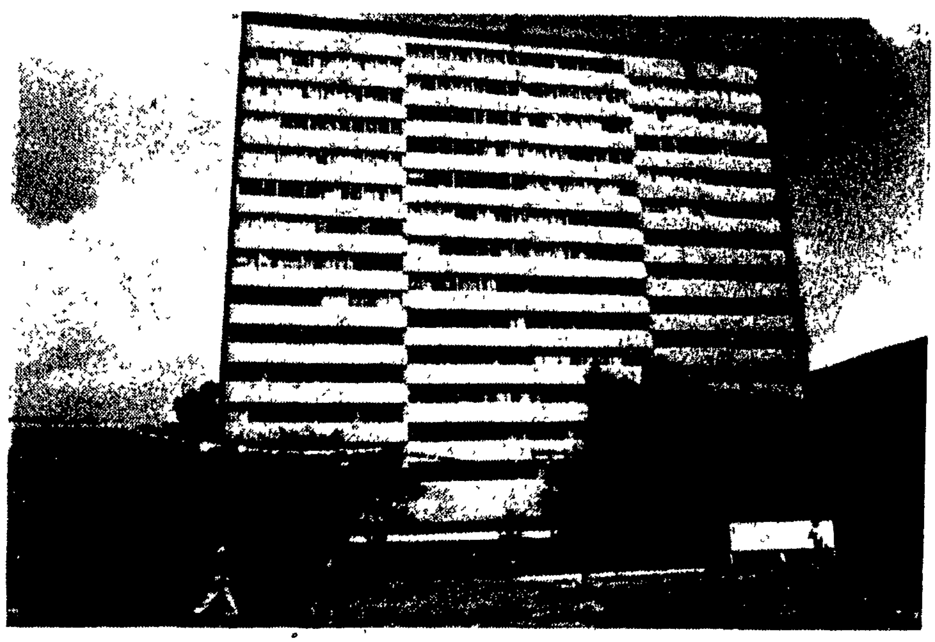


Figure 3.28 Fourteen Storey Apartment Building with Offset Floors which was Damaged in the 1985 Earthquake (Mitchell et al 1986).

3.6 Case Study 5: Twelve-Storey Medical Building at 49 Durango

Figure 3.35 shows the 12 storey structure at 49 Av. Durango. The lateral load resisting system consists of masonry infilled shear walls in the long direction of the building and reinforced concrete frames in the short direction. During the March 14,

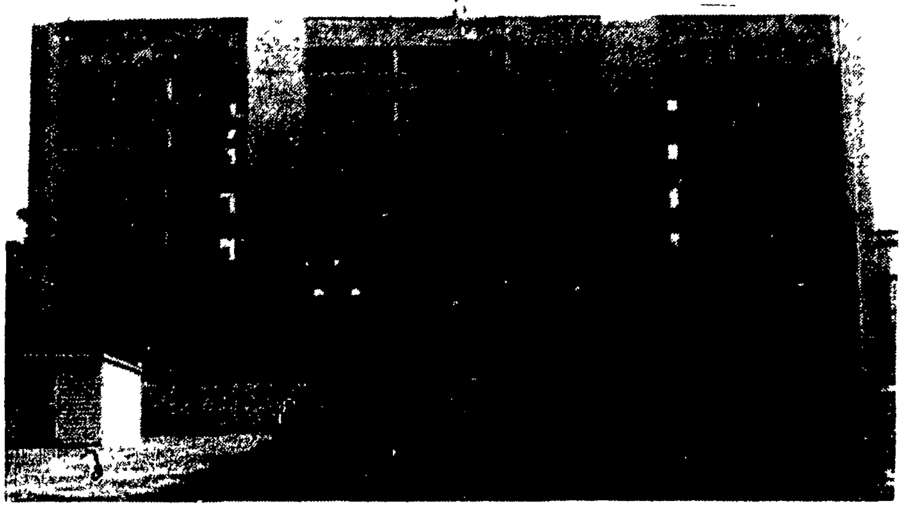


Figure 3.29 Fourteen Storey Building after Removal of 7 Storeys and Strengthening.

1979 earthquake this structure suffered damage in the columns and the beams[9]. The columns were badly cracked in the first three storeys and beams were also cracked by shear and flexural effects. Two different upgrading strategies were studied in order to strengthen the structure. One consisted of slender reinforced concrete shear walls in the interior frames close to the elevator shaft. This solution was not stiff enough to reduce the earthquake forces on the damaged members. Therefore, another solution consisting of vertical and diagonal bracing elements parallel to the extreme frames of the short direction, combined with slab reinforcement, was used. The added steel cross-bracing was attached to the exterior of the building. This bracing consisted of steel channels welded together to form hollow box sections and was connected to the spandrel beams at each floor level by plates and grouted bolts (see Fig. 3.35). The existing slab was reinforced in order to transmit most of the seismic shear force to these new very rigid facades. The existing columns were repaired by adding steel plate jackets and an expansive mortar was used to fill the space between the existing concrete and the plates. Flexural cracks were epoxy injected. Although this structure was in the zone

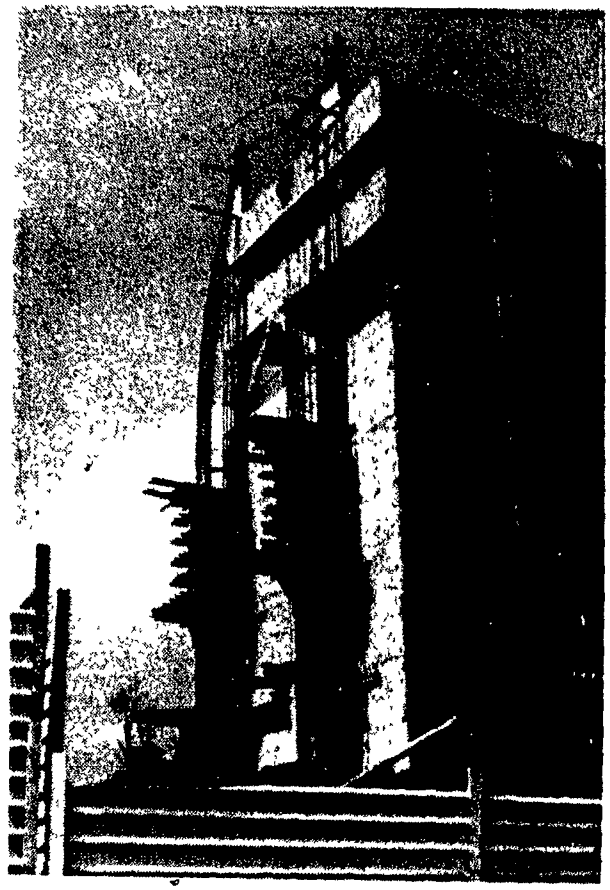


Figure 3.30 Construction of End Shear Walls.

of severe damage during the recent 1985 earthquake (see Fig. 3.1), the building did not suffer any structural damage.

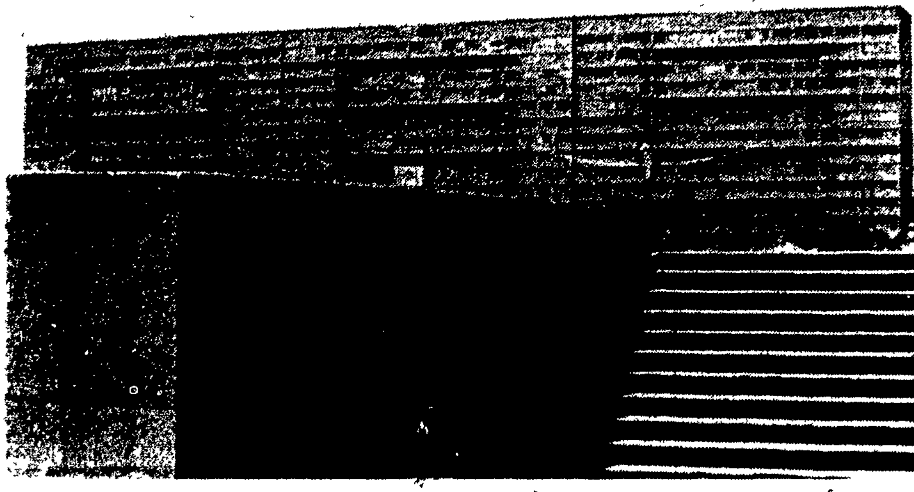


Figure 3.31 The L. Cardenas Apartment Building Damaged in the 1985 Earthquake (Mitchell et al 1986).

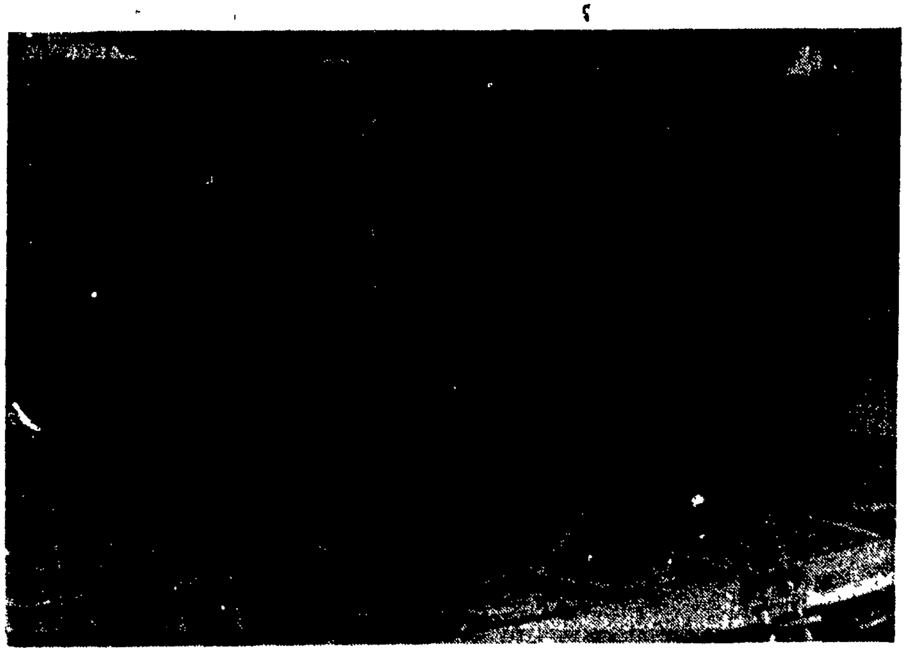


Figure 3.32 Placement of Reinforcement for End Shear Walls to Replace Diagonally Braced Infilled Walls.

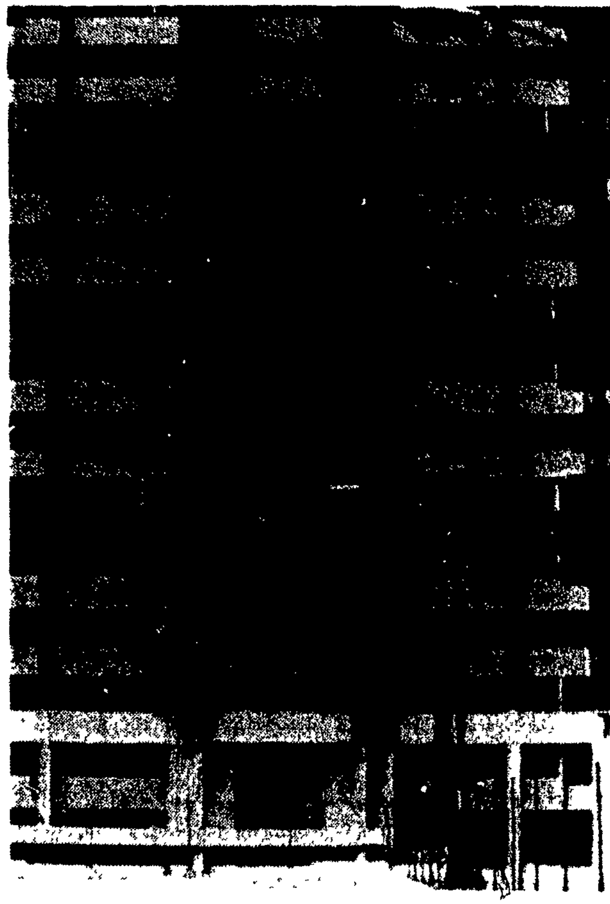


Figure 3.33 Placement of Reinforcing Steel for Added Beams and Columns.

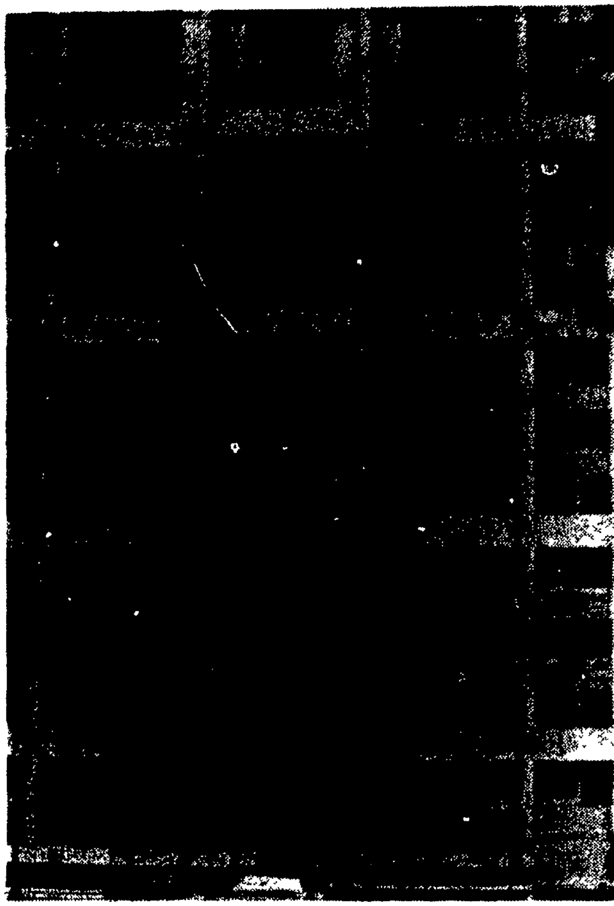


Figure 3.34 View of Structure after Casting of Concrete.

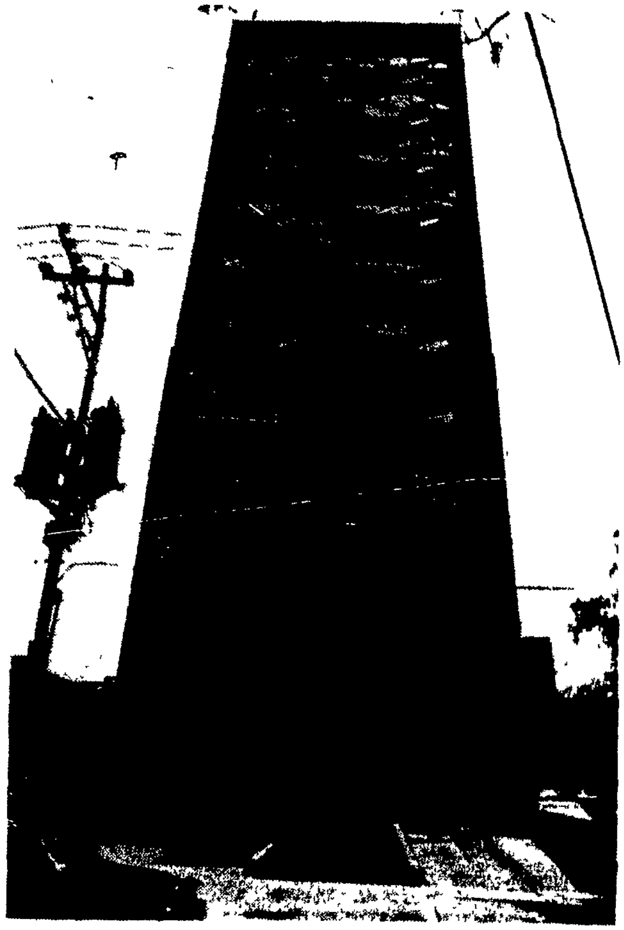


Figure 3.35 Successful Use of Exterior Cross-Bracing to Strengthen and Stiffen a Twelve-Storey Structure.

CHAPTER 4

SEISMIC UPGRADING IN CANADA

4.1 Canadian potential for upgrading and code requirements

There is in Canada a large potential for seismic upgrading of existing structures. Cities such as Vancouver, Ottawa, Montreal and Quebec are located in regions of seismic activity and contain a large number of older structures which are deficient with respect to seismic resistance. Many of these older structures were not designed for seismic loading, while other structures may have been designed for lower values of lateral loads than that required by the current Canadian code regulations for new buildings. The 1985 National Building Code of Canada requires that the design provisions that apply to new buildings also be applied to existing buildings that are being altered. If this requirement is taken literally, no upgrading is required if no modifications are made, while full upgrading is required if only a slight modification is made. This "all or nothing" approach often serves as a deterrent to the upgrading of many older structures due to the difficulties in achieving full compliance with the latest code provisions. This is particularly true for older reinforced concrete structures since the design and detailing requirements of the Code for the Design of Reinforced Concrete Buildings[8] has changed significantly over the years. The impractical and costly nature of providing full compliance with existing codes is a major obstacle to the upgrading of many older structures. A more effective approach in improving public safety would be to target for upgrading particularly hazardous buildings (e.g., unreinforced masonry structures) located in higher seismic zones. These buildings would then be upgraded to satisfy design requirements which would be developed specifically for these types of structures, keeping in mind the impracticality of full compliance and the appropriate level of upgrading. This compromising approach, in contrast to the "all or nothing" approach of the current code, would serve to significantly reduce the seismic hazard of existing Canadian structures. Some engineers in Canada have already adopted such an approach in arriving at a practical solution to the upgrading of existing structures. This is exemplified in the following sections which will illustrate the application of seismic upgrading techniques to existing Canadian structures. An alternative approach modeled on the ATC approach is discussed below.

Although several code approaches for the upgrading of existing structures have been reviewed in Chapter 1, none of them are readily applicable to the Canadian situation. A comprehensive approach to the seismic upgrading of structures in Canada must take into account the vulnerability of some types of construction as well as the large variation of the seismic risk in the country. The current Building Code of Canada[26] already recognizes the widely varying levels of seismic risk by assigning seismic risk coefficients in the form of contours of peak horizontal acceleration and velocity which are used in defining 7 different seismic zones, ranging from 0 (lowest risk) to 6 (highest risk). This zonal distribution is used in the calculation of the base shear in order to provide a quantitative link between the zoning parameters and the desired performance of buildings in earthquakes. The same zonal distribution can be used in the evaluation of existing structures for seismic resistance. The following discussion is an attempt to adapt the ATC approach to a possible regulatory policy for the upgrading of existing structures in Canada.

Fig. 4.1 illustrates, in the form of a flow chart, a procedure adapted to the Cana-

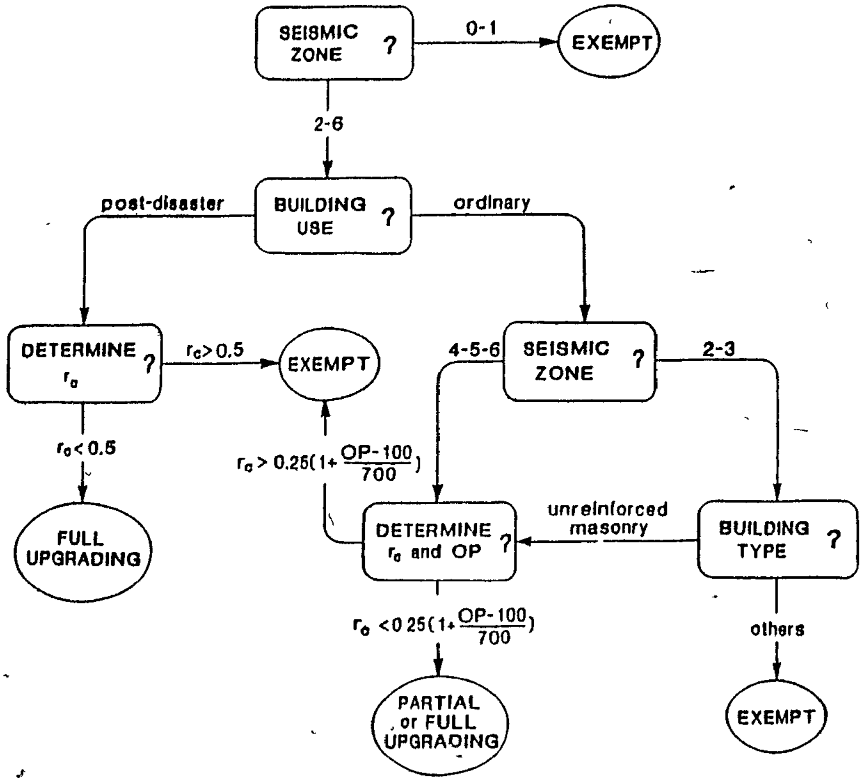


Figure 4.1 Tentative Procedure for the Seismic Evaluation of Existing Structures in Canada.

dian situation for the seismic evaluation of existing structures. In this procedure, existing buildings in regions of low seismicity (zones 0 and 1) are exempt from any seismic upgrading. In zones of moderate to high seismicity (zones 2 to 6), all important existing buildings such as schools and hospitals must be evaluated and upgraded if required. Other existing buildings, if located in zones of moderate seismicity (zones 2 and 3), must be evaluated and possibly upgraded only if the structure corresponds to one of the particularly vulnerable types of construction (i.e., unreinforced masonry or non-ductile reinforced concrete frame), and finally, if located in zones of high seismicity (zones 4, 5 and 6), all existing buildings must be evaluated and possibly upgraded.

The ATC provisions provide the most promising approach for the evaluation of existing structures and for determining the necessary levels of upgrading. If this approach is to be applied to Canada, the following changes are suggested:

(1) Within the framework of Part 4 of NBC, post-disaster buildings and schools are already in a separate category for determining the lateral seismic force level. It is suggested that these buildings could have the same treatment as those buildings classified as Seismic Performance Category D in ATC. Other buildings falling within Part 4 of NBC could be treated as those buildings classified as Seismic Performance Category C.

(2) Although the ATC levels for triggering upgrading seem reasonable, they could be modified if necessary.

(3) In determining the r_c factor for strength, there may be situations where some individual members within the building may be relatively weak and therefore govern the determination of r_c. While this approach is reasonable for structures with brittle members, it may not be appropriate for ductile members. In some situations, it may be possible to account for some redistribution provided that the necessary ductility is present and provided that an appropriate analysis is performed. Care needs to be taken when considering the effects of redistribution, particularly redistribution between columns in a storey level.

(4) In determining the time function for carrying out the necessary upgrading, it is important to relax the ATC provisions in order to reflect the lower seismic risk in Canada. In doing this, a larger α_t and a larger minimum time limit may be chosen. Fig. 4.2 illustrates possible time limits suitable for Canada in which the time function t_x is assumed to have the same form as Equations 1-2 and 1-3 but α_t is assumed to be 15 and the minimum time limit is assumed to be 3 years for post-disaster buildings and schools. For other structures, the minimum time limit is assumed to be 5 years.

(5) In determining the number of occupants for use in assessing acceptable levels of r_c and for calculating t_x, Table 3.1.14.A of NBC can be used. This table gives the area per person in square metres for different types of uses of floor areas.

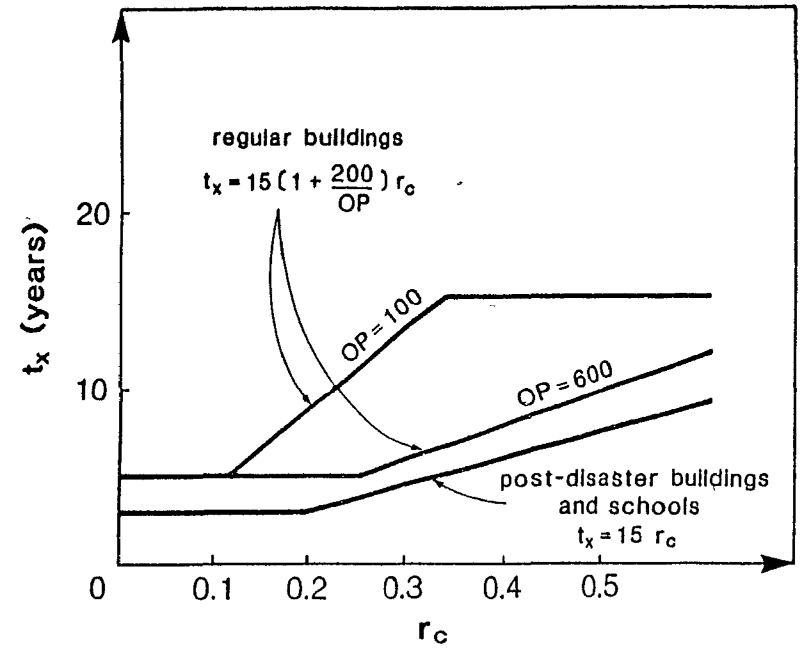


Figure 4.2 Possible Time Limit for the Upgrading of Existing Structures in Canada.

(6) For multi-storey structures, the determination of r_c for all members is an impractical task due to the very large number of members. A different approach is required in order to reduce the computational effort and to take into account the possibility of redistribution of forces and the presence of ductility in the structure. Therefore the risk evaluation approach is not well suited to the evaluation of multi-storey structures. The ductility evaluation approach described in section 1.3(d) is better suited to the evaluation of multi-storey structures since it limits the calculation of the strength coefficients to columns and reduces the number of such coefficients to the number of storeys.

4.2 Case Study 1: Eight-Storey Building at 625 Belmont Street in Montreal

This eight-storey reinforced concrete frame structure was built in 1956. The existing non-ductile concrete frame consisted of columns supporting two way slabs with spandrel beams on the periphery only. The exterior walls were built of masonry. The building houses very sensitive computer and telecommunication equipment. The rehabilitation involved seismic upgrading, provisions for added equipment loads on the roof and the construction of an extension which enclosed new staircases. The construction work was performed while the facility was still in use on a continuous basis. Due to the nature of the building's use, it was subjected to a tight security system. In order to protect the equipment in operation in the facility and to isolate the construction area from the other parts of the building, sealed compartments were built around the construction area and were equipped with ventilation systems to maintain a low pressure inside the compartments, thus reducing the possibility of dust infiltrating the rest of the structure. The total cost of the renovation was \$5.6 million with \$1.0 million for the structural work.

In this case, full compliance with the 1985 Canadian Building Code was required since the building houses telecommunication equipment and hence is classified as an important structure. Furthermore, the costs involved in a damaging earthquake would be very large if the structure had not been upgraded to a high level of performance. The seismic upgrading consisted mainly of the addition of four reinforced concrete shear walls and the addition of tension-compression steel bracing in the extension (see Fig. 4.3). The K factor used in the analysis was 1.3 in both directions. The punching shear resistance of the ground floor slab was also improved in one location where distress of the concrete was apparent (see Fig. 4.4). Four specially designed steel brackets were placed on each face of the column and bolted together to increase the support area of the slab.

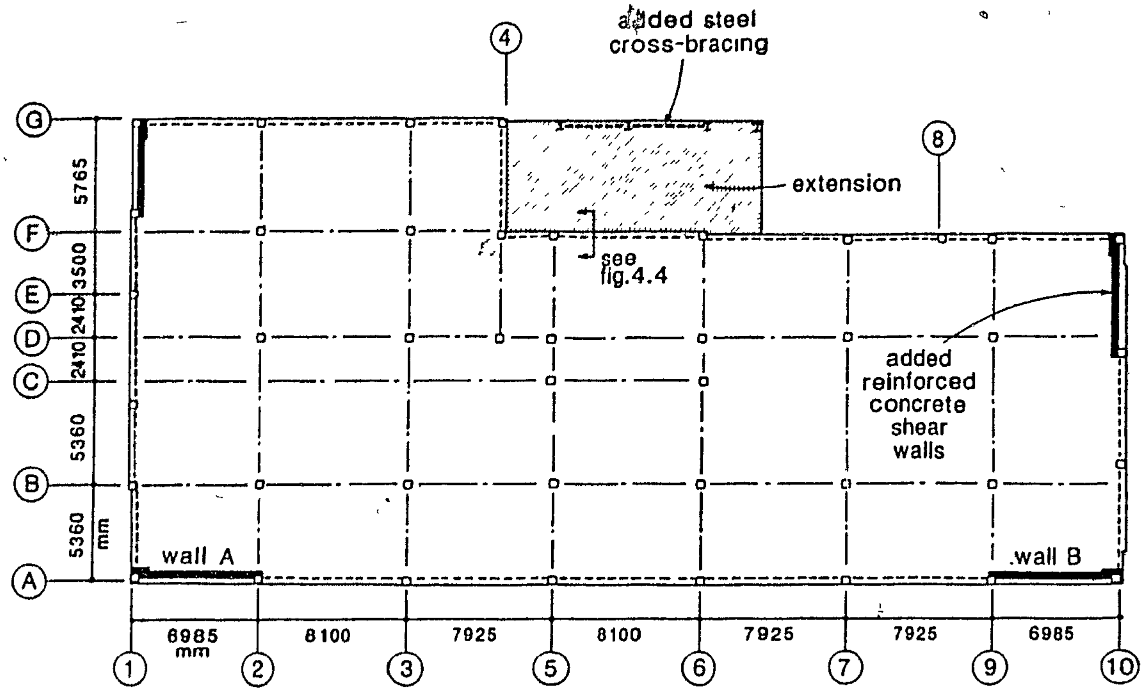


Figure 4.3 Plan View of Eight-Storey Building at 625 Belmont Street in Montreal.

The 230 mm thick shear walls were installed on the inside face of the masonry facing, thus preserving the exterior appearance of the building. At the basement level, the walls were thickened to 530 mm and connected to the existing foundation wall as shown in Fig. 4.5. The shear walls were made composite with the existing frame by the continuous vertical reinforcement of the shear walls passing through the existing concrete slab and by the installation of dowels connecting the shear wall with the existing columns and beams. These dowels were installed at a 300 mm spacing. The continuous vertical wall reinforcement engages the existing concrete slab in diaphragm action while the connection of the wall with the adjacent columns provides stability to the wall when subjected to earthquake loads.

The walls were shotcreted using the existing masonry exterior wall as formwork instead of conventional concreting to reduce the hydrostatic pressure applied to the existing masonry wall. The use of shotcrete reduced the hydrostatic pressures and provided an efficient means of placing the concrete in the congested construction area.

Figure 4.4 Increasing the Punching Shear Resistance of a Column-Slab Joint.

Skilled workmanship was required in order to maintain a uniform thickness of concrete.

The location of the added concrete shear walls involved the obstruction of some windows. This situation was resolved through the use of "false windows" which served no other purpose but to preserve the original exterior appearance of the building.

The new steel braced extension is an important contributor to the lateral load capacity of the structure and therefore must be linked with the remaining parts of the building. The horizontal diaphragms of the extension and of the existing building were connected using special details capable of transmitting large shear forces. The connection is provided by "Hilti" bolts drilled into the existing concrete spandrel beam

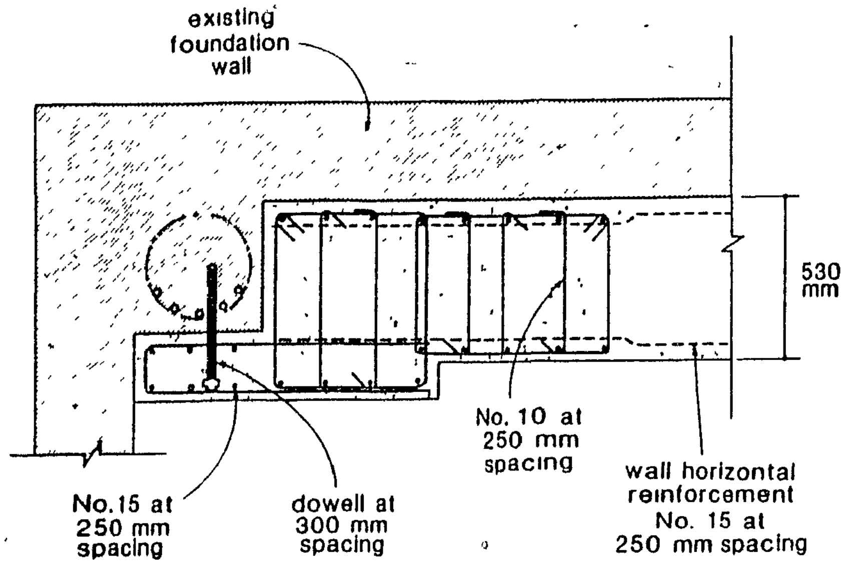


Figure 4.5 Added Reinforced Concrete Shear Wall.

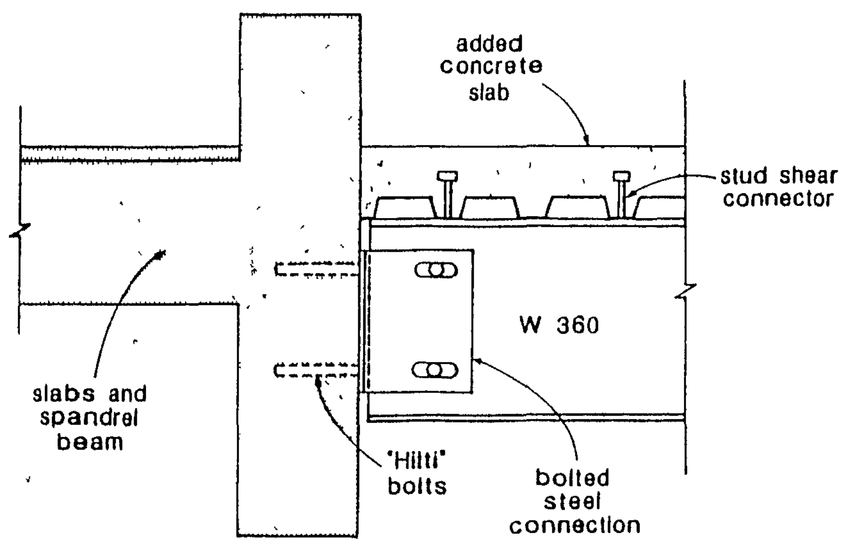


Figure 4.6 Shear Connection Between Added and Existing Diaphragms.

and welded to a steel connector which was then bolted to the new steel beams (see Fig. 4.6). The new concrete slab was made composite with the steel braced frame of the extension with stud shear connectors. The thickness of the new slab in the extension increases in the lower floors in order to resist the larger shear forces.

The existing pile foundations were found capable of resisting the added loads due to

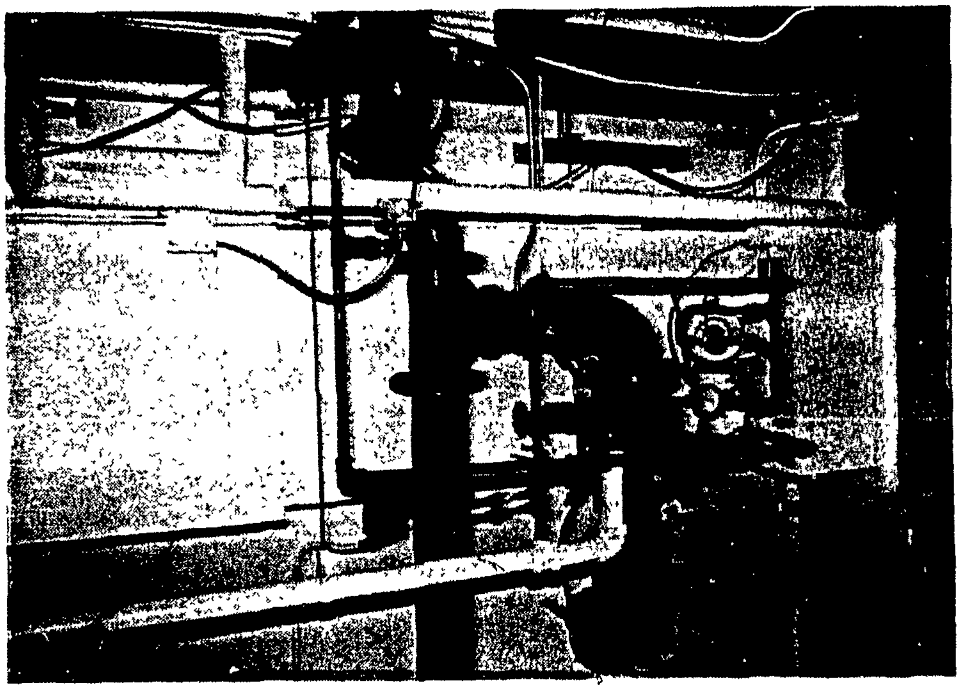


Figure 4.7 View of an Added Beam and Existing Mechanical Facilities.

the new shear walls. However the foundation wall joining the added shear walls needed to be strengthened so that the loads being induced by the added shear walls could be distributed to adjacent piles. Two heavily reinforced horizontal beams supporting the added shear walls (walls A and B) were built at the top and bottom of the existing foundation wall to provide continuous foundation beams over piles located at column lines 1-2-3 and 7-9-10. As often the case in upgrading projects, the presence of existing mechanical facilities requires unusual detailing of the added members to accommodate these facilities. Fig. 4.7 shows the "cut-out" in one of the beams along the foundation wall imposed by the presence of existing mechanical facilities.

4.3 Case Study 2: Seven-Storey Office Building at 550 Beatty Street in Vancouver

The seven-storey structure consisted of unreinforced masonry bearing walls on the periphery with an interior timber column and flooring system. This office building is typical of many older masonry structures and is joined to adjacent buildings by two party walls on each side. The strengthening against lateral loads was achieved by the construction of new reinforced concrete shear walls which also served as stair cases and elevator shafts. The location of the added shear elements is shown in Fig. 4.8. In the direction of the party walls, the lateral force is resisted partly by the new shear walls and the existing masonry walls. The K factor used in this direction is 2.0. In the other direction, the added shear walls were designed to resist the total lateral force, hence the K factor used in this direction is 1.3.

Foundation work included new footings for the added shear walls. Some column footings were replaced and enlarged. An interesting feature of this project is the strengthening of the masonry wall corners at the foundation level (see Fig. 4.9). This area is particularly sensitive as the corner will be subjected to concentrated forces from the earthquake load and may fail due to the lack of out-of-plane resistance and foundation capacity. The strengthening of the corners was achieved by adding new footings under the corner area and two perpendicular reinforced concrete walls which were connected to the masonry wall with dowels. The added perpendicular walls were also connected together through continuity of the horizontal reinforcement.

A diaphragm was created by the addition of a reinforced concrete topping over the existing timber deck. The concrete topping is 75 mm thick and is reinforced with a 152 x 152 x 18.7 mm welded wire fabric. The diaphragm is connected to the shear resisting elements through continuity of the reinforcement. Similar details are shown in Section 2.2. New reinforced concrete spandrel beams were added to the two building facades. These beams are connected to the added concrete topping by extending the

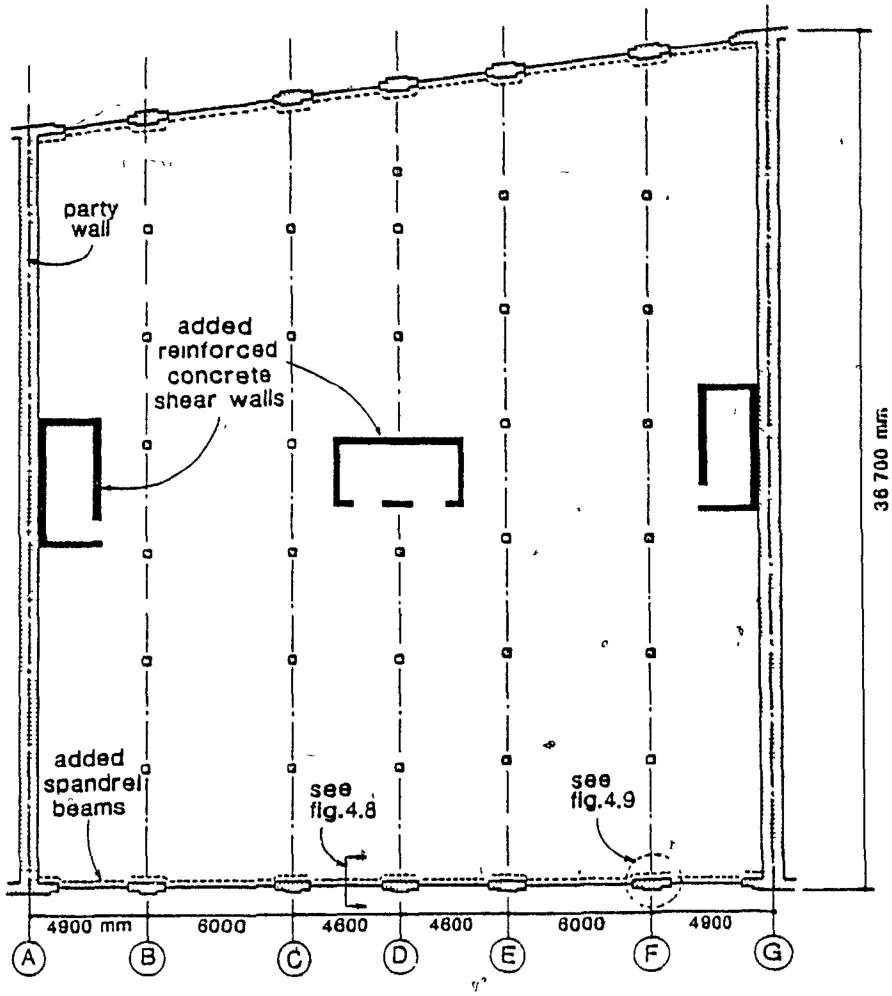


Figure 4.8 Plan View of Seven-Storey Office Building at 550 Beatty Street in Vancouver.

welded wire fabric into the beam section (see Fig. 4.10). The vertical load coming from these beams is transmitted into the existing masonry pilaster by extending the top longitudinal beam reinforcement in the slab region around the pilaster and by encasing the perpendicular timber beam into the concrete spandrel beam. A plan view of the reinforcement distribution around the existing pilaster is shown in Fig. 4.11.

The beam-column connections of the timber flooring system were improved as shown in Fig. 4.12. Two steel plates were bolted to the connecting beams in order to tie these two beams together. Knee braces were also added in certain locations to provide some moment resistance to the connection.

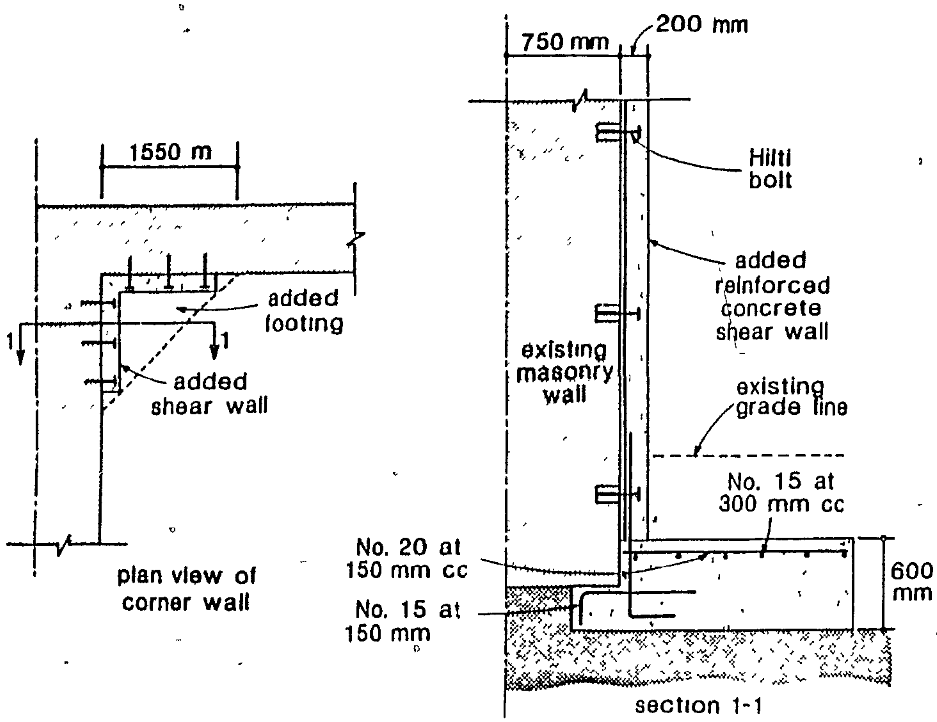


Figure 4.9 Strengthening of the Corner of a Masonry Wall.

This structure was upgraded to resist the full lateral load required by the 1975 National Building Code of Canada. But, as is often the case in the upgrading of older buildings in dense city areas, it was impractical to satisfy the requirements for the separation from adjacent buildings (N.B.C. 75 4.1.9.2 (5)) and the existing brickwork was not reinforced (N.B.C. 75 4.1.9.3 (4)).

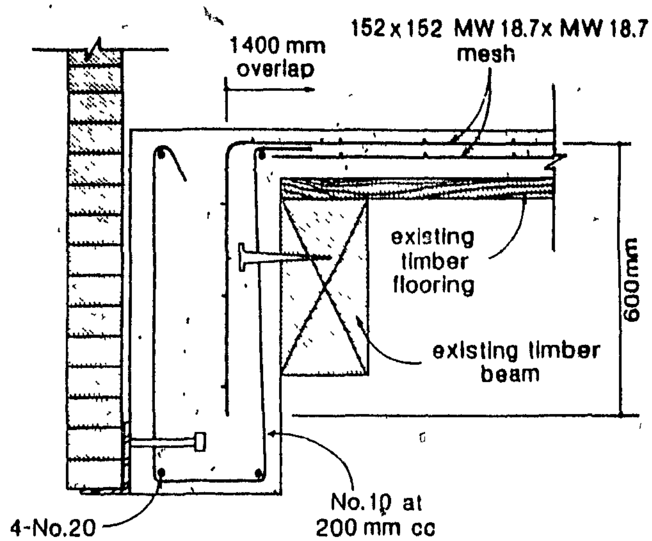


Figure 4.10 Cross-Section of Added Spandrel Beam.

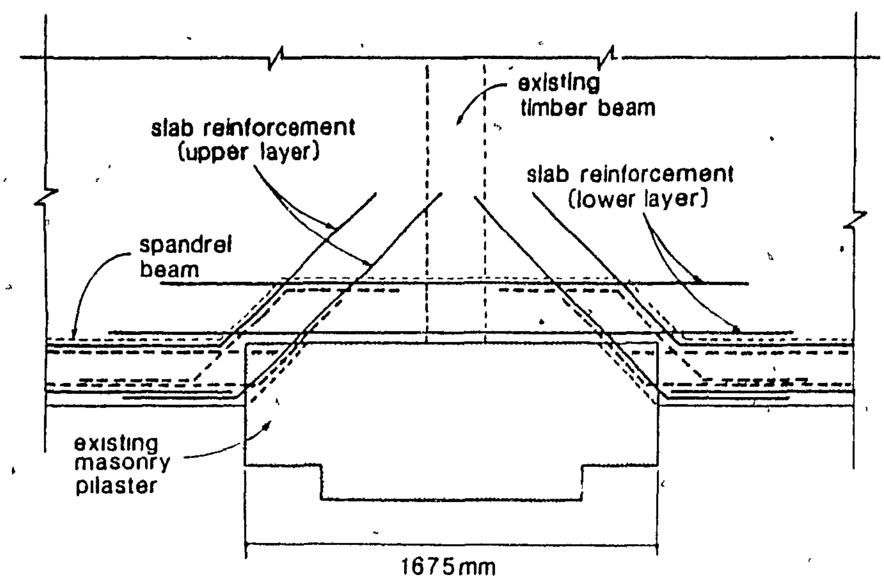


Figure 4.11 Plan View of Reinforcement Distribution Around Existing Masonry Pilaster.

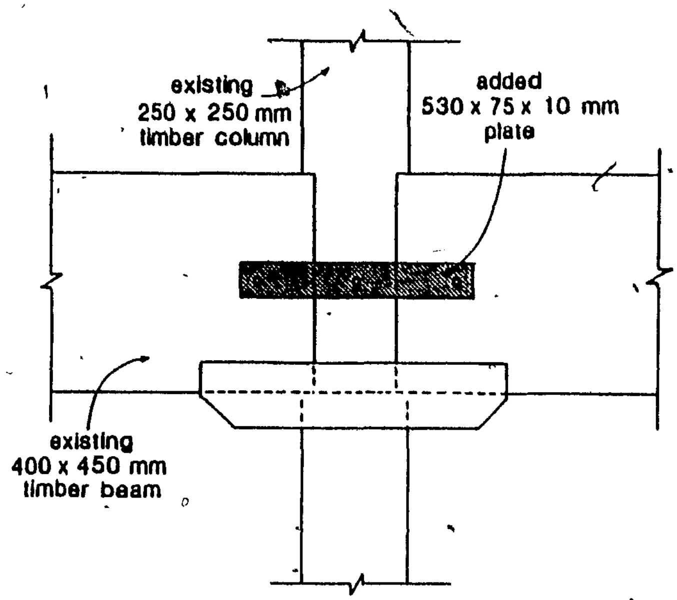


Figure 4.12 Strengthening of Timber Beam-Column Connections.

4.4 Case Study 3: Eight-Storey Building at the Corner of Hasting and Setmour in Va

This eight-storey office building is composed of two parts, phase I and phase II (see Fig. 4.13). Phase I was built in 1910 and consist of a non-ductile reinforced concrete frame with no lateral load resisting elements such as shear walls or bracing. Phase II was built in 1956 and consist of a reinforced concrete frame with shear walls in both directions located around staircases and elevator shafts. This arrangement led to a very eccentric structure since most of the stiff elements were concentrated in one region of the structure. Furthermore, the two parts had no common bearing wall and were separated by a 125 mm construction joint infilled with brick veneer. The two parts could therefore react independently to earthquake loading and consequently pound against each other.

The upgrading of this structure aimed at reducing the eccentricity and eliminating the possibility of pounding between the two parts rather than achieving full compliance with the current National Building Code. Reinforced concrete shear walls were added in both directions of the older part of the structure where very little stiffness was provided. The horizontal diaphragms of the two parts were connected at every floor

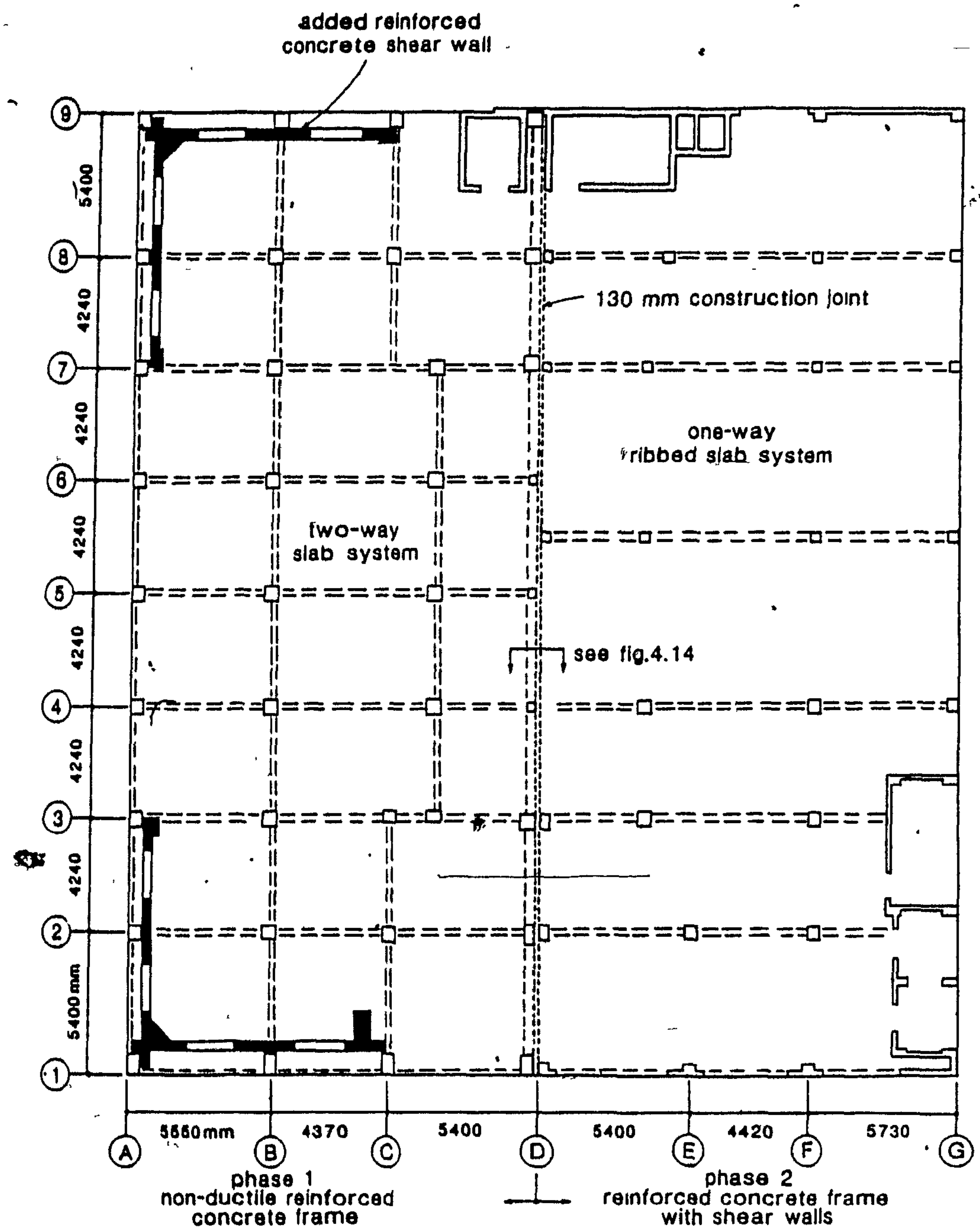


Figure 4.13 Plan View of Eight-Storey Office Building at the Corner of Hasting and Setmour in Vancouver.

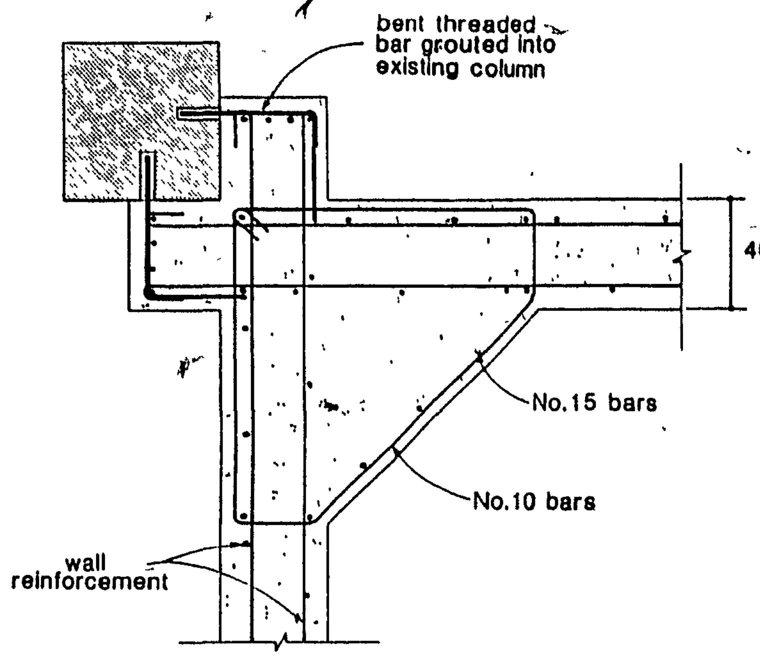


Figure 4.14 Detail of Connection Between Added Shear Wall and Existing Column.

level so that they would act as one entity. The added shear walls in phase I combined with the existing shear walls in the other part of the structure were designed to resist 75% of the seismic forces required by the 1980 N.B.C. (N.B.C. 80 4.1.9.).

The new shear walls were added on the inside of the structure just beside the edge beam. This allowed the vertical wall reinforcement to be continuous through the existing concrete slab. The connection of the added shear wall with the existing columns was achieved by threaded bars bent as shown in Fig. 4.14.

Where shear walls were added, the existing footings had to be enlarged. Fig. 4.15 illustrates in a plan view the configuration of the added footing area with the existing configuration of column footings. New footings were installed around and in between the existing column footings in order to increase the bearing area. Deep beams were used to connect added footings on each side of existing column footings.

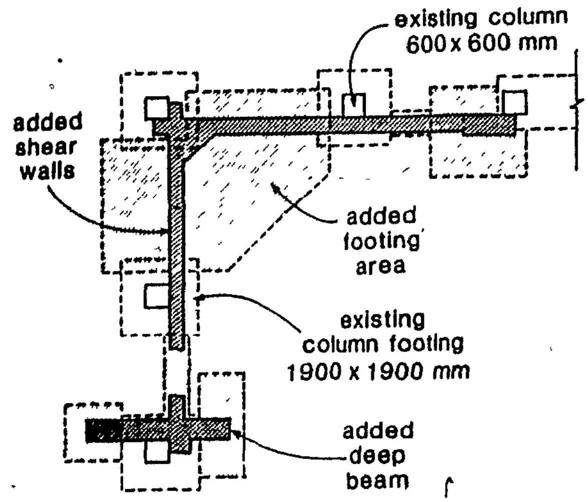


Figure 4.15 Plan View Showing the Configuration of Added Footing Area.

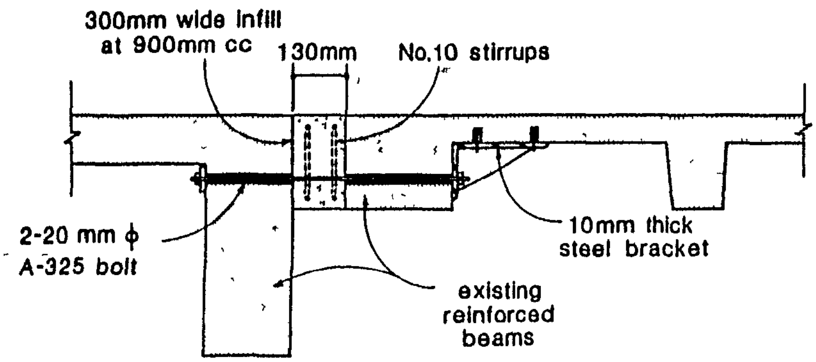


Figure 4.16 Cross-Section of Diaphragm Connection.

The two parts of the building were connected by filling the construction joint and by connecting the two parts at each floor level (see Fig. 4.16). At intervals of 900 mm, the existing topping was broken and any existing brick veneer removed in order to open and expose the separation. Two holes were drilled to connect the two edge beams with bolts. At each connection location, a 300 mm long portion of the construction joint was infilled with concrete and reinforced with stirrups.

4.5 Case Study 4: Three-Storey Office Building at 101 Water Street in Vancouver

This three-storey building was modified as well as upgraded. The original structure consisted of unreinforced masonry walls on the periphery and timber columns and beams on the interior. The foundation consisted of a mixture of column footings and timber piles with concrete pile caps. One of the exterior walls was entirely rebuilt from the foundation up to the roof level using reinforced concrete blocks.

The foundation system was upgraded for seismic loading by the construction of a series of grade beams joining the piles caps and the column footings. The added grade beams can be seen on Fig. 4.17. Also a large footing area was added at the location of the new reinforced concrete core and column footings were added at the location of new steel columns. The plan view illustrates the complexity of this upgrading project which has been accentuated by the diversity of the existing foundation system. Due to architectural and site constraints, many types of materials and techniques were used. This is typical of many upgrading projects where renovation and possible change of use of the facility is taking place simultaneously with the seismic upgrading of the structure. In a situation such as this one, where different types of foundation systems or piles had been used in the original structure, the addition of grade beams play an important role in limiting differential movements of the foundation.

A concrete topping, with an average thickness of 50 mm, reinforced with a welded wire fabric was added on top of the timber decking of the three floors. The new diaphragm was connected to the existing exterior masonry walls with dowels embedded in the topping and grouted into the existing masonry wall.

The roof connections were braced as shown on Fig. 4.18 to provide some moment resistance at the top of the peripheral walls and to permit the diaphragm action developed by the floors.

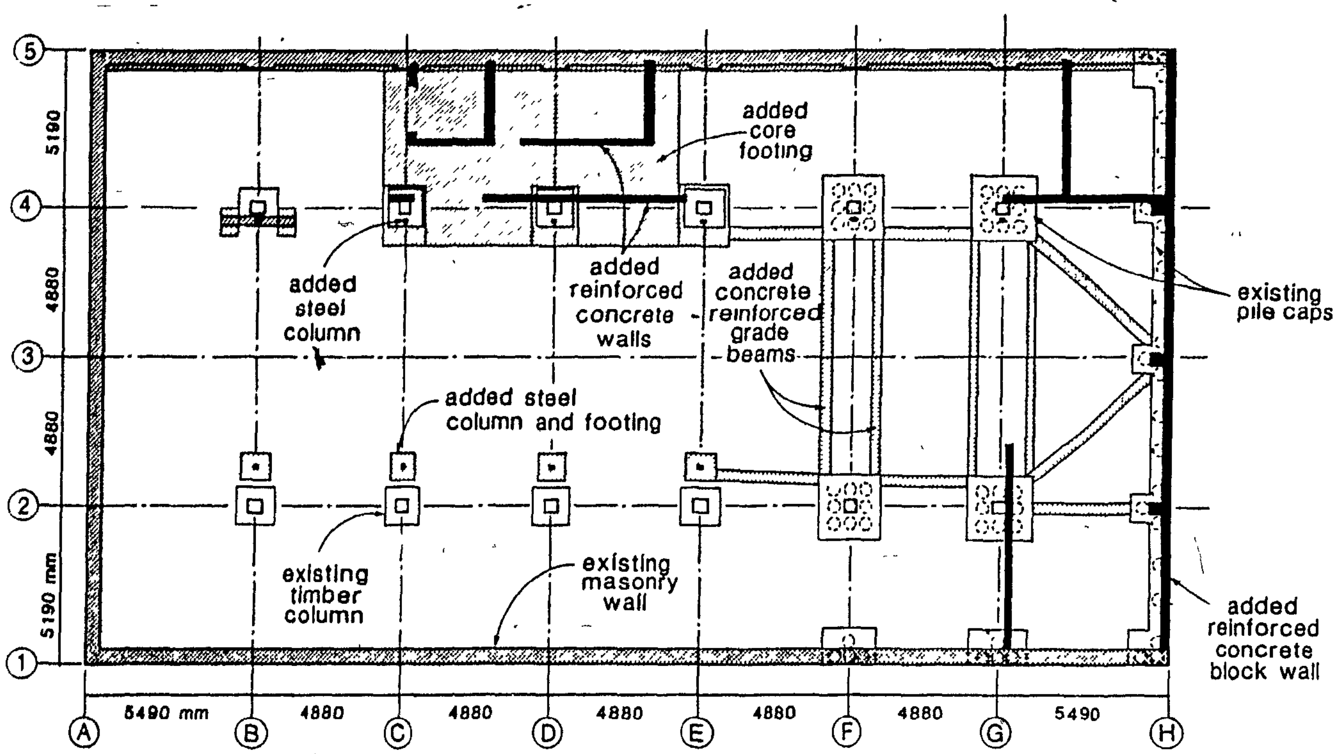


Figure 4.17 Plan View of Three-Storey Office Building at 101 Water Street in Vancouver.

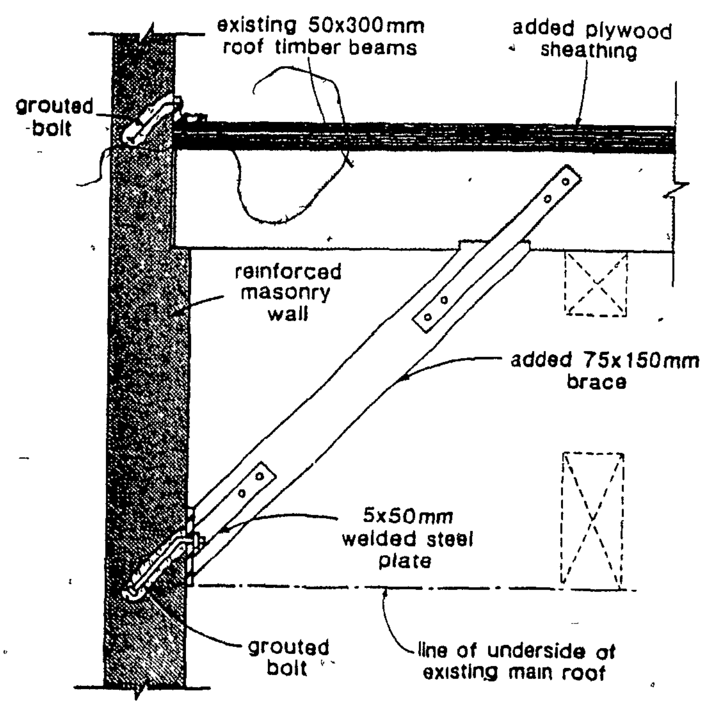


Figure 4.18 Bracing of the Roof Connections.

CHAPTER 5

DESIGN CONSIDERATIONS

5.1 Load History Analysis

Most solutions to the problem of upgrading an existing structure involve the addition of new material to existing structural elements. In some situations, the load history of the structure plays an important role in determining the capacity of the upgraded element. It must be determined whether the initial strains in the loaded structure prior to upgrading have a significant influence on the capacity of the upgraded member. Their influence is usually to reduce the capacity of the member. If this reduction is significant, then the initial strains must be taken into account in the design of the upgraded member.

As an illustration of a load history analysis, the design of an upgraded reinforced concrete column is demonstrated in this section. Two upgrading techniques will be considered. The original column section is described in Fig. 5.1a. The first repair technique (see Fig. 5.1b) consisted of increasing the column dimensions from 400×400 mm to 600×700 mm and by providing additional longitudinal and transverse reinforcement.

The first step in evaluating the capacity of the strengthened column is to determine the initial strains in the column prior to the upgrading. It is assumed for the

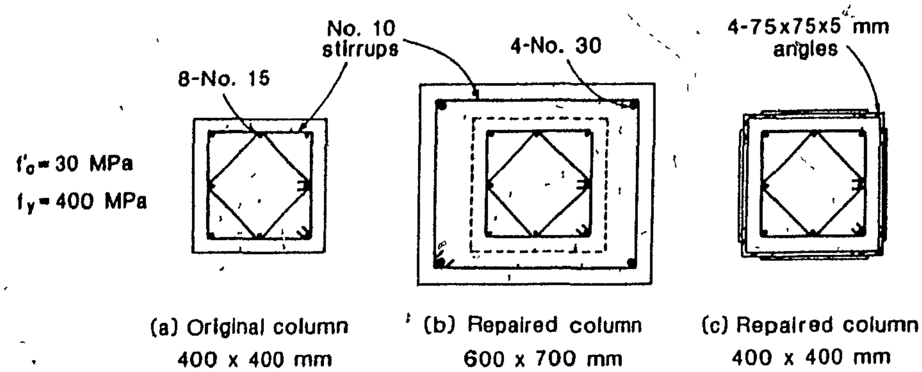


Figure 5.1 Cross-section of Column and Strengthening Techniques.

purpose of this example that the original column is upgraded 25 years after its original construction. The analysis is described below:

(1) Calculation of long term material properties of the original column: The long term properties of the original concrete column were calculated using the methods described in Reference 6. The 28-day compression strength of the concrete is assumed to be 30 MPa. Creep is accounted for by using a reduced initial stiffness, $E_{c,eff}$ (the tangent modulus is used). The creep coefficient, $\phi_{(t,t_i)}$ (t = 25 years, t_i = 7 days), is equal to 2.082, thus giving a reduced initial stiffness of 9 773 MPa. The corresponding peak strain, ϵ'_c, is -6.139 $\times 10^{-3}$. The shrinkage strain, ϵ_{sh} (t = 25 years), is assumed to be -0.4 $\times 10^{-3}$.

(2) Calculation of the strain distribution prior to strengthening: For the purpose of this example, a service axial load of -1800 kN is taken as a reasonable axial load for this typical interior column. In this analysis, the material strength reduction factors used will be equal to unity (i.e., $\phi_c = \phi_s = 1.0$). The distribution of the total strain, ϵ_c, is computed by using the long-term material properties in the computer program PLANE[6] and by performing a "layer-by-layer plane section analysis". The results are illustrated in Fig. 5.2.

(3) Calculation of the long term material properties of the upgraded column: For the

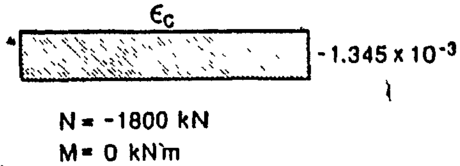
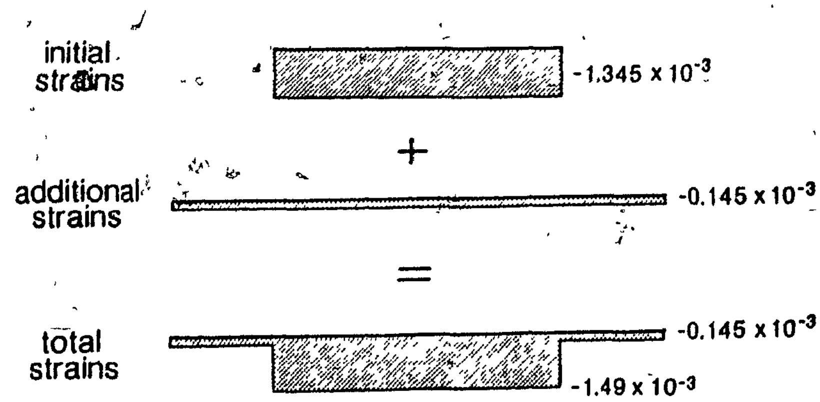


Figure 5.2 Strain Distribution Prior to Strengthening.

purpose of this example, it is assumed that the earthquake will occur 5 years after the column has been strengthened. During this period, the creep and shrinkage of the old concrete was calculated to be negligible. However, the added concrete (with $f'_c = 30$ MPa) will undergo a certain amount of creep and shrinkage. For the new concrete, the creep coefficient, $\phi_{(t,t_i)}$ (t = 5 years, t_i = 7 days), is 1.729. The reduced initial stiffness, $E_{c,eff}$, is 11 040 MPa and the peak strain, $\epsilon'_{c,eff}$ is -5.435 $\times$ 10^{-3}. The shrinkage strain, ϵ_{sh} (t = 5 years), is -0.250 $\times$ 10^{-3}.

(4) Calculation of the strain distribution prior to the earthquake: As the new concrete shrinks, it will be subjected to some tensile strains since the shrinkage is restrained at the interface between the old and the new concrete. The old concrete will be forced into some additional compressive strains in order to preserve the internal equilibrium. By using the long term material properties of both the old and new concrete in the computer program PLANE and performing a layer-by-layer analysis, the additional concrete strains due to shrinkage of the new concrete is obtained. The results are shown in Fig. 5.3. The distribution of the total strain prior to the earthquake is obtained by adding the initial strain distribution obtained in step (2) to the additional strain distribution due to the shrinkage of the new concrete and the creep of the new and old concrete (see Fig. 5.3).

(5) Short term analysis of the upgraded column: Due to the nature of the earthquake loading, short-term material properties were used in the analysis. The bending moment capacity of the column is determined for various values of axial loads by a layer-by-layer analysis. In this analysis, the difference in initial strain between the old and the new



$\epsilon_c = \epsilon_{sh} + \epsilon_{co} + \epsilon_{cf}$

Figure 5.3 Strain Distribution Prior to the Earthquake.

concrete is taken into account. Figure 5.4 shows a stress-strain diagram illustrating the state of strain and stress of a concrete layer prior to the earthquake.

The initial stress, f_{ci}, is found from the initial strain, ϵ_c, using the long-term secant modulus as shown in Fig. 5.4a.

Thus

$$f_{ci} = E_{c,eff}\,(\epsilon_c - \epsilon_{sh})\,. \tag{5-1}$$

The pertinent strain components for the new and old concrete are shown in Fig. 5.5. The resulting initial stresses, f_{ci}, are shown in Fig. 5.6.

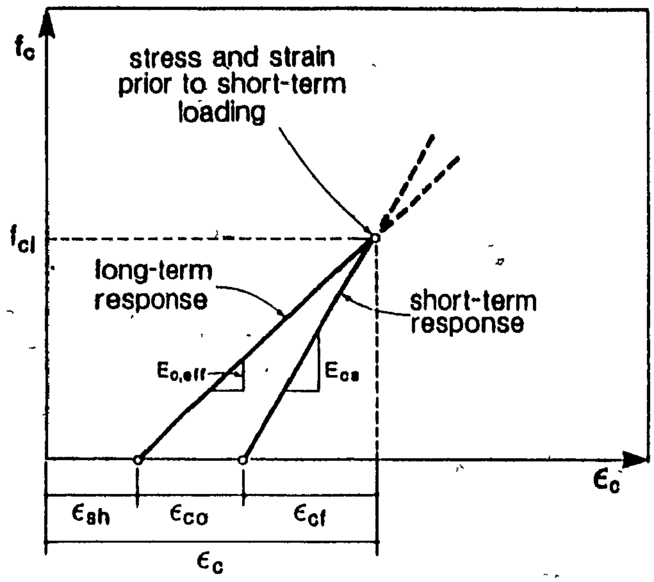
The short term response of the concrete will follow a curve shifted from the origin by a distance $(\epsilon_{sh} + \epsilon_{co})$ and with an initial slope equal to E_c, the short term secant modulus.

The total strain offset, $\epsilon_{sh} + \epsilon_{co}$, can be determined from Fig. 5.4a as:

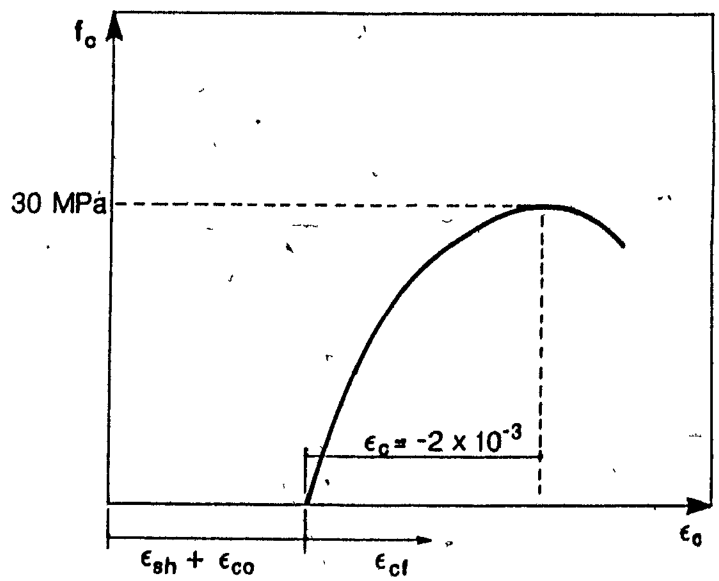
$$\epsilon_{co} + \epsilon_{sh} = \epsilon_c - \epsilon_{cf} = \epsilon_c - \frac{f_{ci}}{E_c} \tag{5-2}$$

These strain components for this example are shown in Fig. 5.7.

The resulting short-term, parabolic stress-strain relationship is shown in Fig. 5.4b. The strain caused by stress is found from:



(a) Determining strain offset for short-term stress-strain relationship



(b) Short-term stress-strain relationship

Figure 5.4 **Accounting for Load History of a Concrete Layer.**

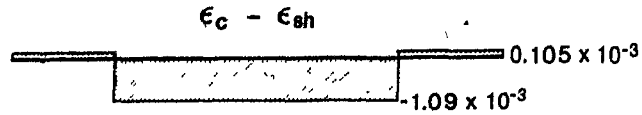


Figure 5.5 Distribution of strain ($\epsilon_c - \epsilon_{sh}$).

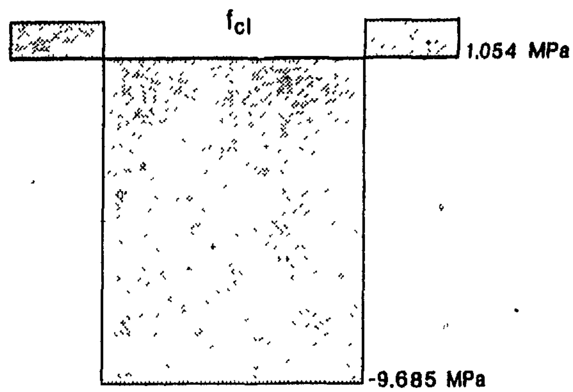


Figure 5.6 Initial Stress Distribution.

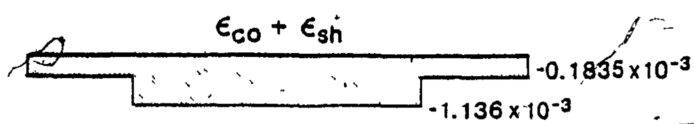


Figure 5.7 Distribution of ($\epsilon_{co} + \epsilon_{sh}$).

$$\epsilon_{cf} = \epsilon_c - \epsilon_{co} - \epsilon_{sh} \tag{5-3}$$

In the analysis of the section, we set the initial strains equal to the total strains, ϵ_c, for each layer and we set the shrinkage strain equal to the strain not causing stress, $\epsilon_{co} + \epsilon_{sh}$ (strain offset), in the computer program PLANE. A layer-by-layer analysis is then performed for the column section with different levels of axial loads using the short-term parabolic stress-strain relationship for the concrete (see Fig. 5.4b). In this analysis, the design strength reduction factors $\phi_c = 0.6$ and $\phi_s = 0.85$ are used. The results presented as a P-M interaction diagram are shown in Fig. 5.8. The same analysis was performed without taking into account the load history of the upgraded column. The diagram shows a very small difference between the two analyses and would suggest that, in this case, the more complex load history analysis is not necessary. However, it must be noted that the influence of the initial strains depends on their magnitude. In this example for the original column, the axial load taken was relatively small and no moment was present. A larger initial axial load together with an initial bending moment would contribute to increasing the influence of the initial strains on the capacity of the upgraded column. It is therefore suggested, as a conservative precaution, that slightly smaller strength reduction factors may be applied to the old concrete if the load history of the member is not taken into account in the analysis. These smaller strength reduction factors should also take into account the possible long-term deterioration of the old concrete.

Similarly, the analysis of another strengthening technique used on the same original column was performed. This technique consist of adding at the corners of the column section steel angles welded together with horizontal batten plates. The details are shown in Fig. 5.1c and the results of the analysis are also presented in Fig. 5.8. The same area of added longitudinal steel as in the previous example was used. Al-

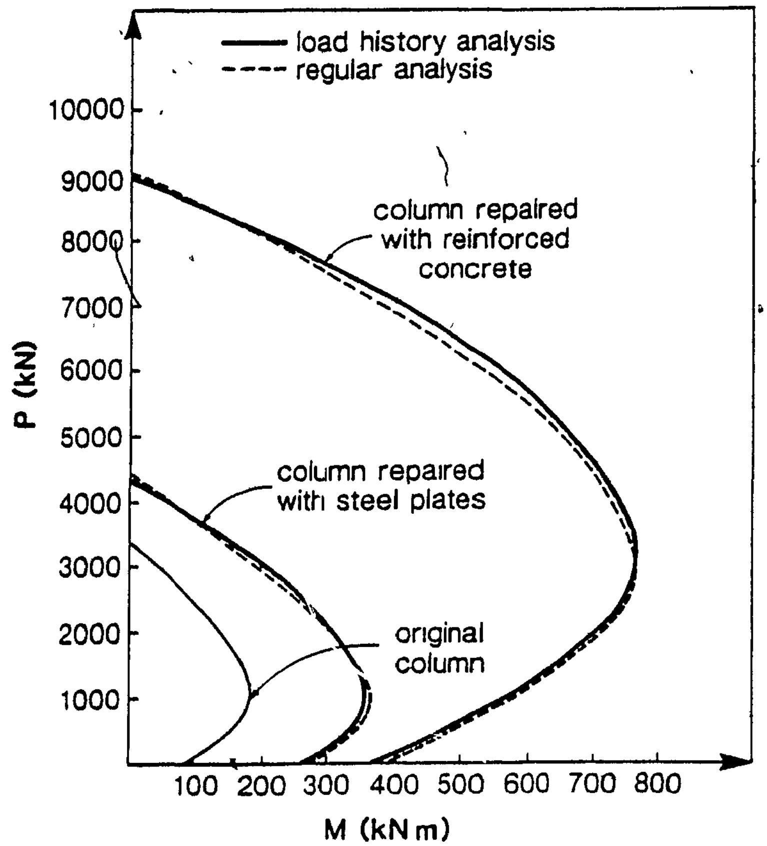


Figure 5.8 P-M Interaction Diagram for Short Term Response.

though the resulting capacity is significantly improved from the original column for both strengthening techniques, the first example shows a much larger increase in moment and axial load capacity. The added concrete area together with the larger lever arm of the added steel reinforcement accounts for the larger capacity resulting from the first strengthening technique. In situations where the occurrence of a brittle shear failure is a concern (short-column effect) and where a large increase in moment capacity in not desired, it seems apparent that the second strengthening technique would be more suitable as it provides a significant increase in confinement while limiting the increase in moment capacity.

5.2 Special Design Considerations for Added Shear Walls

A convenient method for increasing the lateral load strength and stiffness of a reinforced concrete frame structure is to add a shear wall between existing columns (e.g. see Fig. 3.6 and Fig. 3.16). This method involves the enlargement of the columns and the addition of longitudinal and transverse reinforcement. The new web is reinforced with both uniformly spaced vertical and horizontal reinforcing bars. All vertical reinforcing bars pass through holes in the floor slabs and are lapped to ensure continuity. The horizontal bars in the web are anchored into the confined region of the new concrete around the existing column. Fig. 5.9 illustrates how the tension in the horizontal web bars gets transferred to the end columns. For the design of this important connection, the strut and tie model is a useful tool. The flow of compressive stresses in the concrete is represented by compressive struts and the tension in the reinforcement is represented by tension ties (see Fig. 5.9a). As can be seen from Fig. 5.9b, the flow of forces can be represented by a simple truss of known geometry. The anchorage reinforcement is designed in order to resist the maximum capacity of the horizontal wall reinforcement. In this example, the horizontal wall reinforcement consists of two No. 10 bars at 300 mm spacing. The anchorage reinforcement is designed to transmit the nominal yield strength of the horizontal wall reinforcement (i.e., $\phi_s = 1.0$). The strut and tie model is analyzed as a truss subjected to external loads as shown in Fig. 5.9b. The resulting forces in the truss elements are shown in Fig. 5.9c. The anchorage reinforcement is chosen to resist these forces. The maximum tension tie force is 53.3 kN in members GH, HI and IJ. As can be seen from Fig. 5.9d, there are two legs of the No. 10 bars which resist this force if both the anchorage reinforcement and the column ties are spaced at 300 mm. Hence the tension tie capacity of members GH, HI and IJ is $0.85 \times 2 \times 100 \times 400 = 68\ kN$. Tie members FK, KL and LA have only one leg and thus have a capacity of 34 kN. Therefore, the No. 10 column ties are spaced at 150 mm such that a capacity of 68 kN is provided over a height of 300 mm.

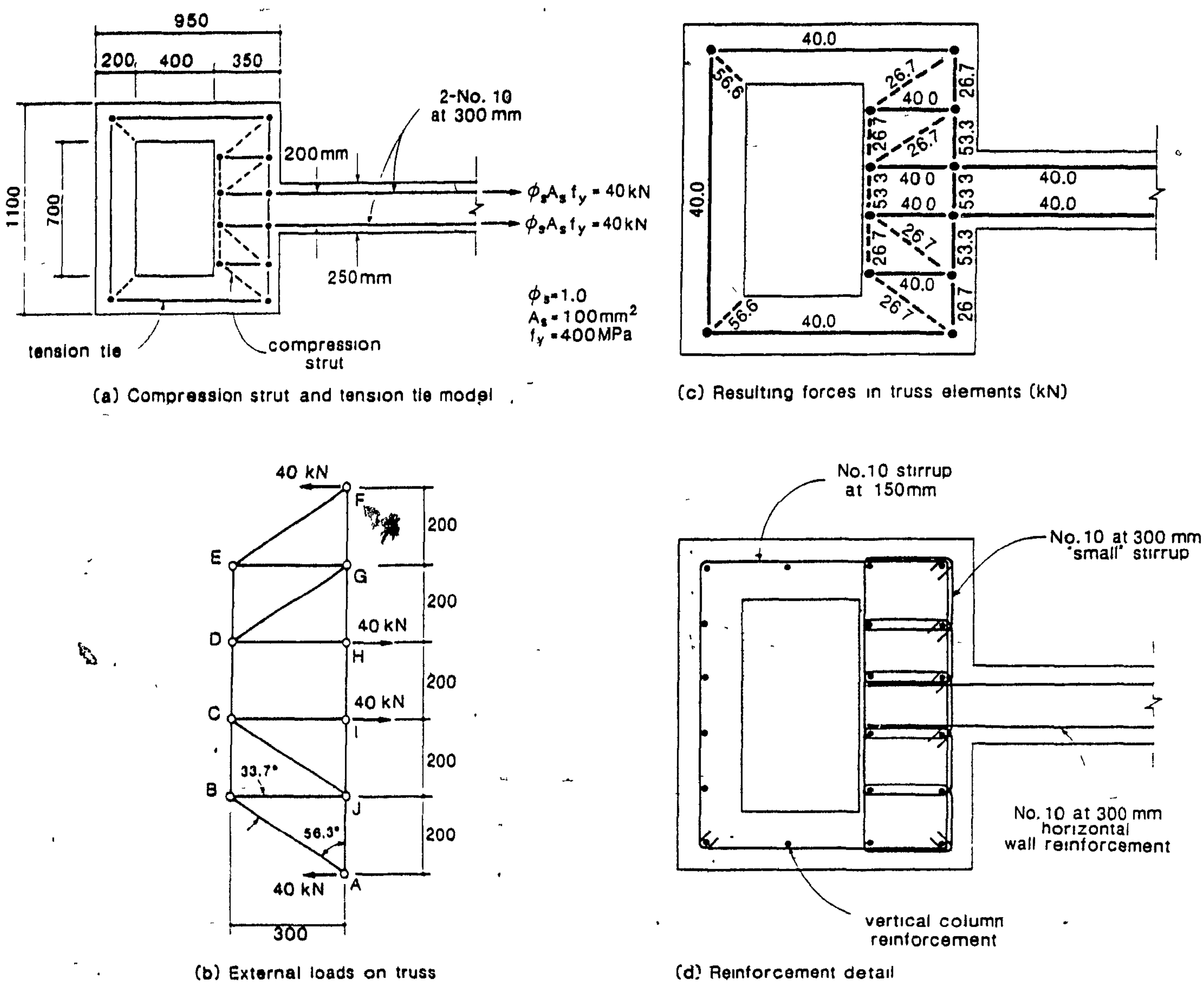


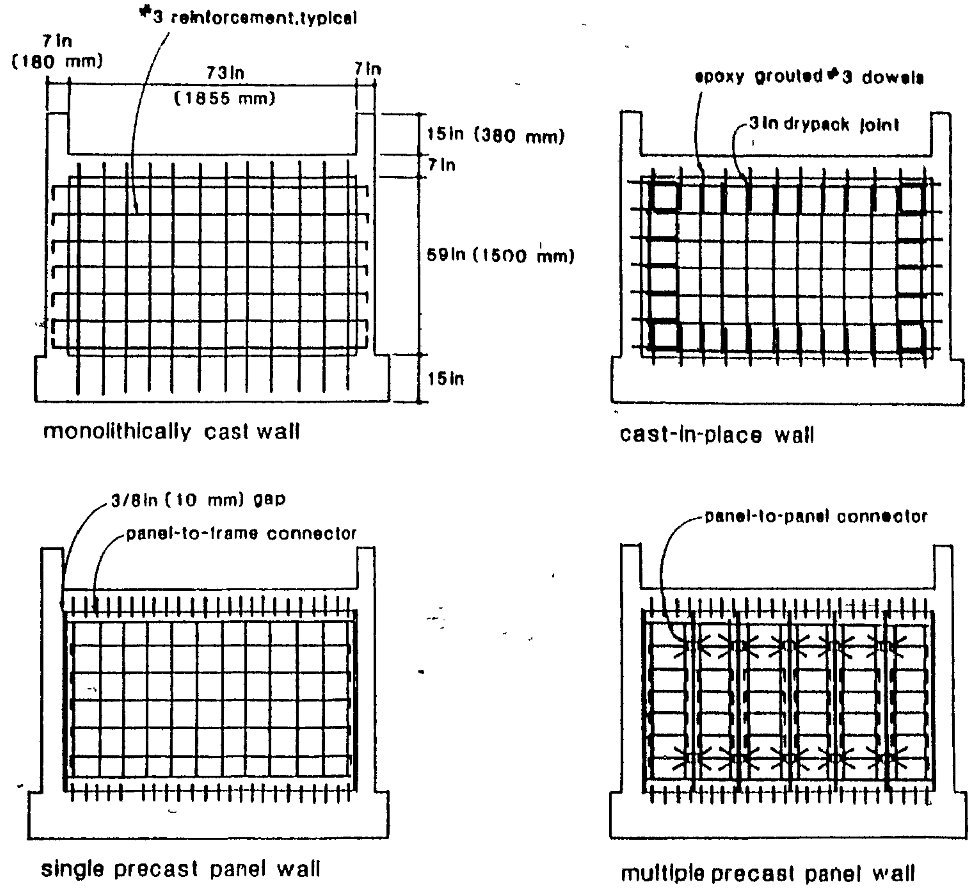
Figure 5.9 Design of Anchorage Reinforcement for Infilled Shear Wall.

5.3 Survey of Experimental Data

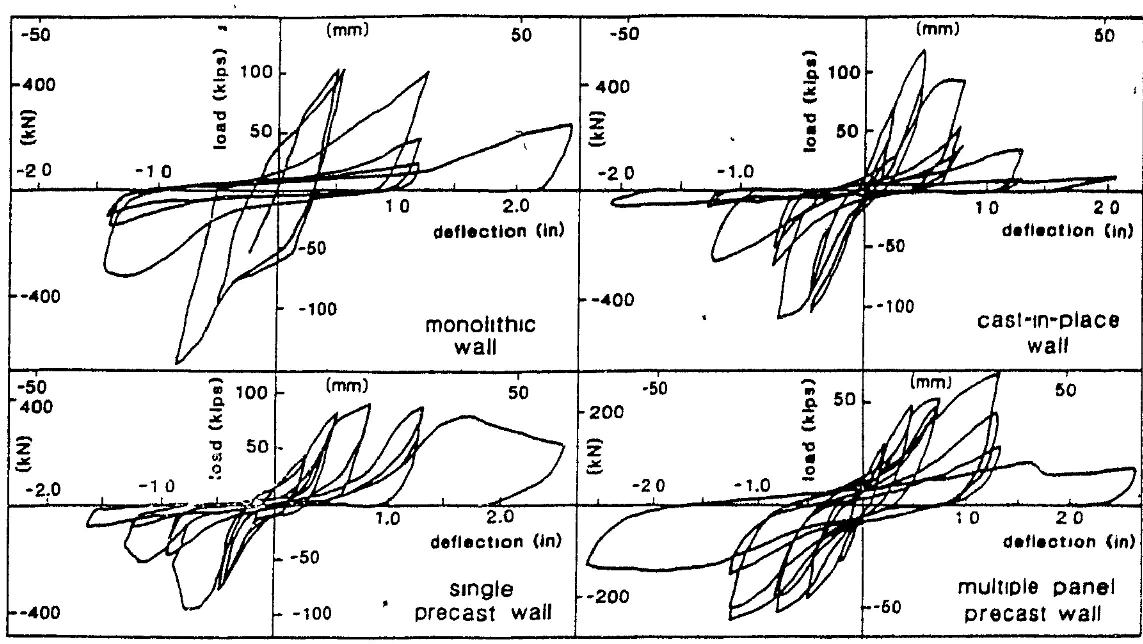
As discussed previously, the available guidance for the design engineer faced with the task of upgrading an existing structure is very limited. Pertinent experimental data could be crucial in situations where innovative construction techniques are considered. This section presents briefly some of the more recent experimental results which address directly the upgrading of existing structures.

5.3.1 Infilled Walls Within Existing Reinforced Concrete Frames

A series of tests were performed by Kahn and Hanson[14] on different techniques for infilling reinforced concrete shear walls within existing frames. Three different techniques were tested and their behaviour was compared with that of a monolithically cast wall. The details of the infilled walls and the test results are illustrated in Fig. 5.10. The cast-in-place infilled wall had about the same maximum capacity as the monolithic wall. Failure in the cast-in-place wall was initiated by deterioration of the joint between the top of the cast-in-place wall and the bottom of the top beam. The deterioration of this wall was more rapid than that of the monolithic wall. The full panel and the multiple panel precast infilled walls had lower maximum capacities with the single panel having a capacity of about 3/4 and the multiple panel about 1/2 of the monolithic wall specimen capacity. Although the ductility of these two specimens was substantially greater than the other specimens, the stiffness was substantially lower. The total energy dissipated by the monolithic wall was about twice that of the other specimens which all had similar levels of energy dissipation.



a)Reinforced patterns for infill walls



b) Load deformation response

Figure 5.10 Test Results for Different Techniques for the Infilling of Walls Within Existing Frames[14].

5.3.2 Short Columns

A series of tests were performed by Jirsa and Wyllie[31] on different techniques for the strengthening and repair of short columns. Four specimens were tested for reversed lateral loading. The details of the column strengthening and the test results are illustrated in Fig. 5.11. Specimen 1-1 was tested, repaired and then retested as specimen 1-1R. Specimen 1-1 and 1-1R exhibited shear-dominated failures while specimen 1-2 ad 1-3 exhibited flexural-shear failures. Both the strengthened and the repaired columns exhibited greater ductility than the original column. Supplementary crossties which were drilled through the existing column did not significantly increase specimen strength nor stiffness but were beneficial in delaying strength and stiffness deterioration under repeated cycles to high drift levels. Specimen 1-1R had much greater lateral stiffness and strength than the original specimen, however because the original column was damaged it has a slightly smaller strength and stiffness as compared with Specimens 1-2 and 1-3.

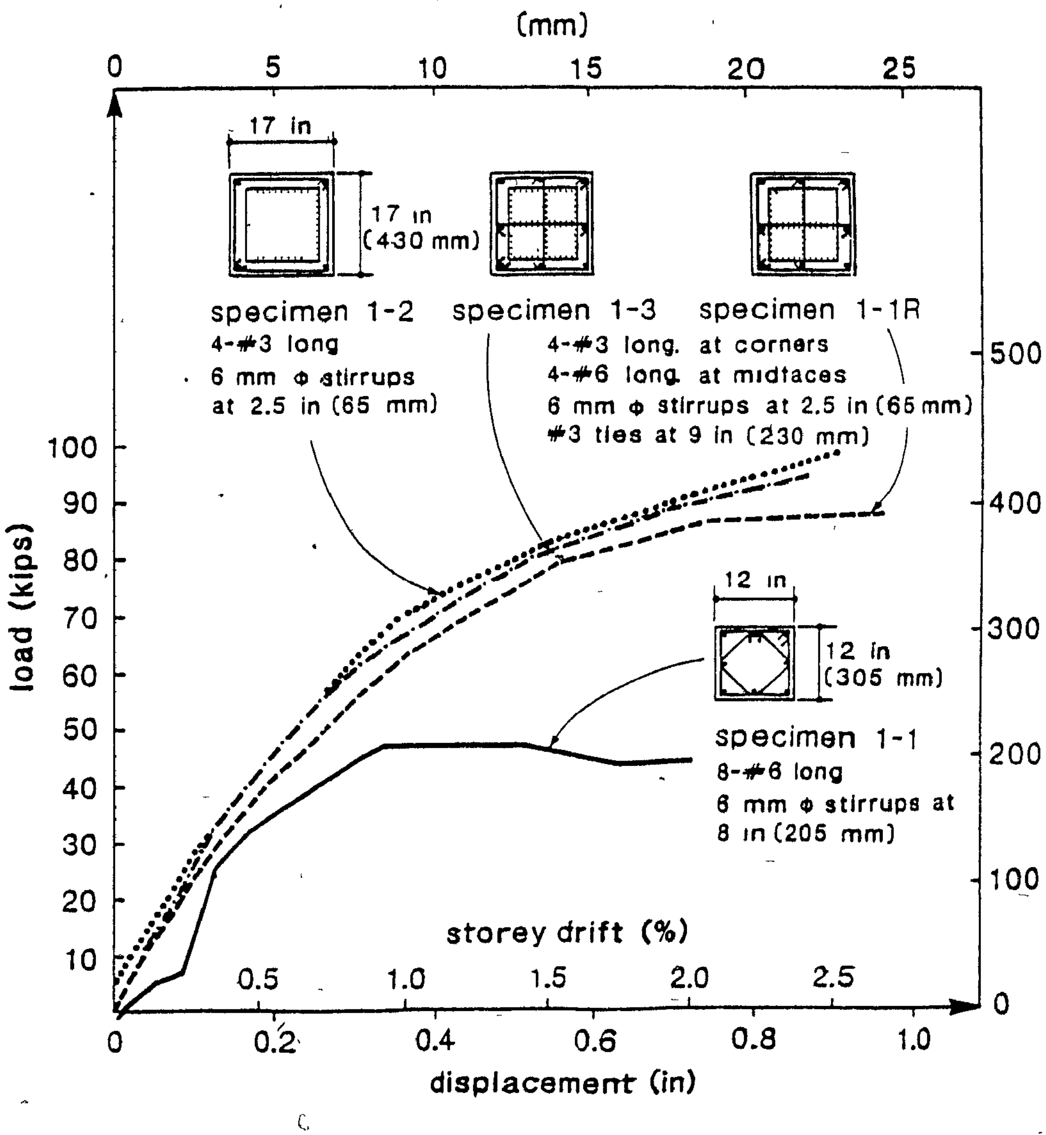


Figure 5.11 Test Results for Different Techniques of Strengthening and Repair of Short Columns[31].

5.3.3 Reinforced Concrete Frame Sub-Assemblages

Two strengthening techniques were used in tests performed by Jirsa and Wyllie[31]. The original prototype was a 2/3 scale model and consisted of a frame with deep spandrel beams and short columns between the floor levels. The columns were not heavily reinforced in order to simulate the unfavorable "strong-beam, weak-column" system. The first strengthening technique consisted of converting the columns into stiff and strong elements by adding new concrete "column-walls" at each column location. Details are shown in Fig. 5.12a. Under load, the strong columns forced inelastic action into the beams and converted the structure from a "strong-beam, weak-column" system to a "strong-column, weak-beam" frame. The second strengthening technique consisted of using an exterior steel frame with lateral resistance provided by diagonal steel members. Details are shown in Fig. 5.12a. Both systems performed very well under lateral loading. Figure 5.12b shows an envelope curve of the lateral force-drift response of the original and the strengthened frames. The increase in lateral capacity is readily apparent. However, the second technique involved some minor problems. The main problem was the difference between the stiffness of the concrete frame and the diagonal steel braces. In addition, attachment of the steel members posed detailing problems. Because the diagonal steel elements introduced a vertical force component into the columns, they were strengthened with channel sections anchored to the corners of the column.

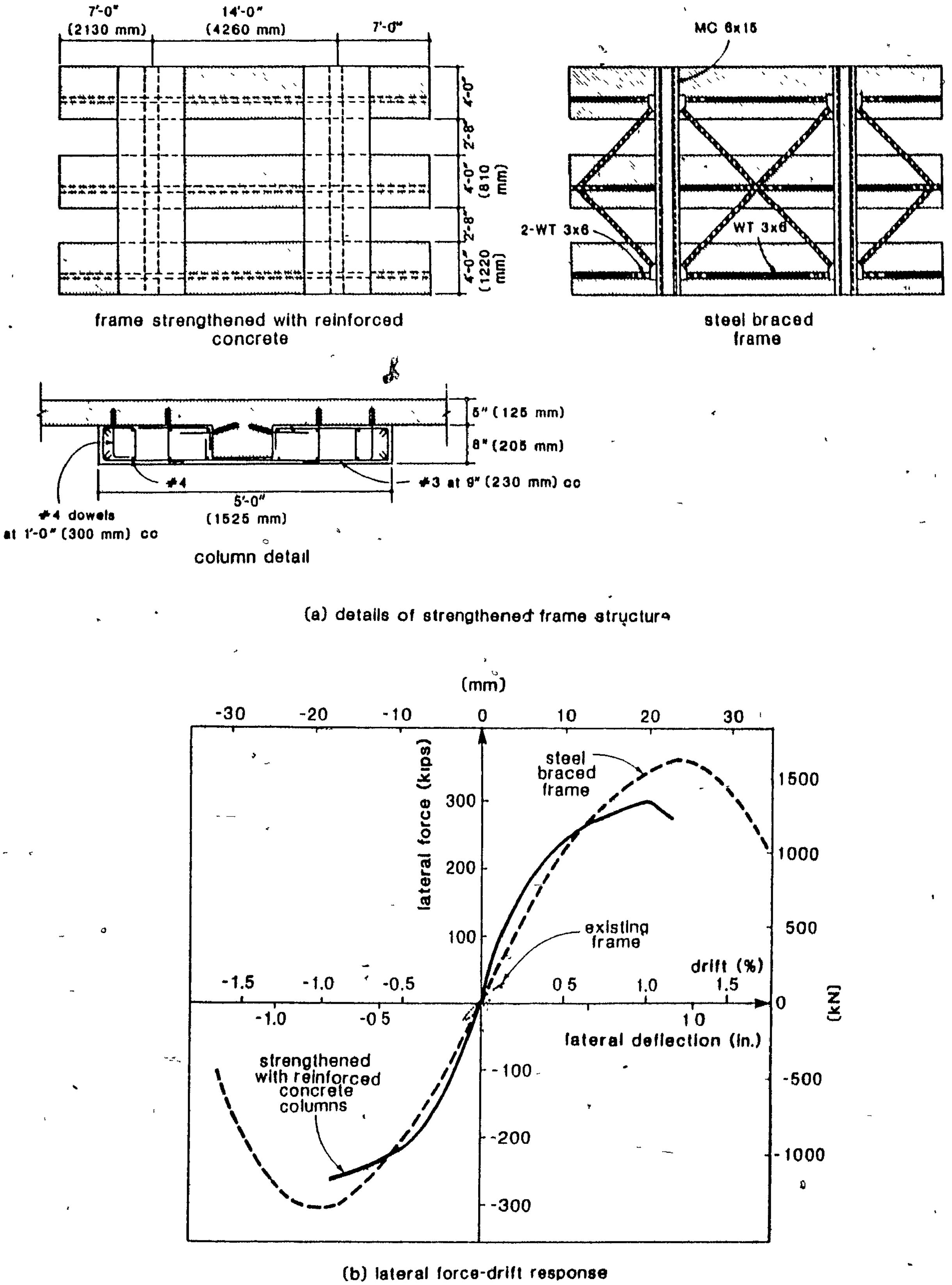
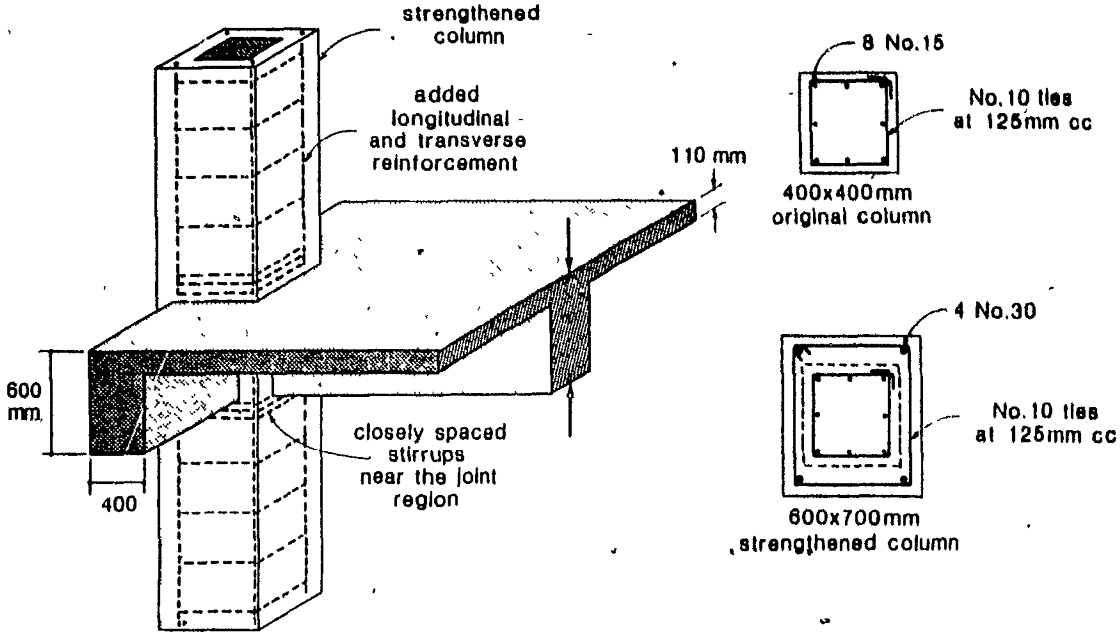
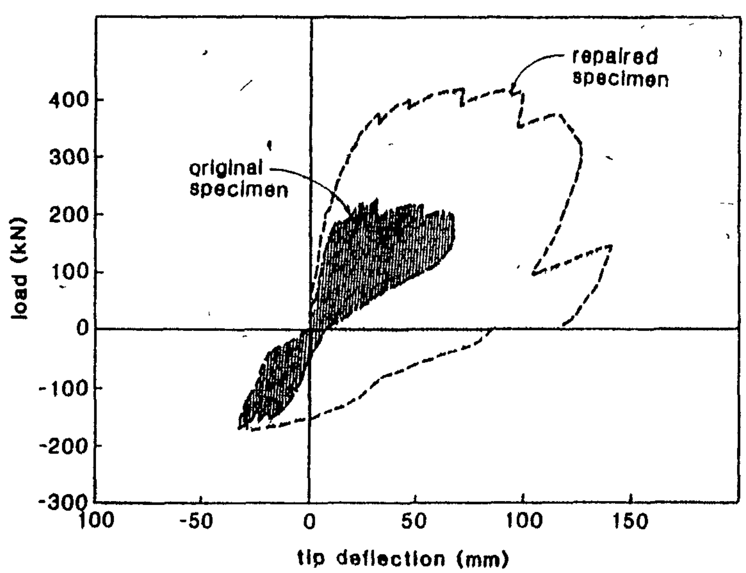


Figure 5.12 Tests results for Different Techniques of Strèngthening a Frame[31].

Another test was performed by Castele and Mitchell[5] which investigated the influence of strengthening a column on the behaviour of the beam-column joint. The original specimen consisted of a 400 × 400 mm edge column with two 400 × 600 mm connecting spandrel beams, one 400 × 600 mm main beam and a 110 mm slab (see Fig. 5.13a). The original specimen was tested by applying reversed cyclic loading at the tip of the main beam. The specimen failed by flexural hinging of the column and by shear in the joint region. A second identical specimen was built which was then strengthened by increasing the column dimensions to 600 × 700 mm and by adding longitudinal and transverse reinforcement. The joint region was strengthened by placing closely spaced column stirrups near the top of the slab and the bottom of main beam. The strengthened specimen was tested and showed a much greater capacity than the original specimen (see Fig. 5.13b). The failure mode was modified and the specimen developed a plastic hinge in the beam near the column face. This allowed much larger plastic deformations, ductility and energy absorption capacity. These test results illustrate the potential benefits of increasing the column and joint capacity in order to transform a "weak-column, strong-beam" system into a more adequate "strong-column, weak-beam" system.



(a) Specimen



(b) Response envelope

Figure 5.13 Test performed by Castele and Mitchell[5].

5.3.4 Shear Interfaces

Jirsa and Wyllie[31] investigated the strength and load deflection characteristics of the interface connection between old and new concrete typical of that used in repair and strengthening of existing reinforced concrete structures. Each specimen consisted of a base block simulating an existing reinforced concrete column with a wall cast against the base block at a later stage. Several concrete surface preparation techniques including sandblasting, chipping, shear keys and dowels were considered. The specimens were subjected to repeated lateral shear loads acting in the plane of the concrete interface at various load levels until failure. Some of the results are presented in Fig. 5.14. The following conclusions resulted from these tests:

(1) The shear strength increased with the dowel embedment length and the number of dowels but not in a linear manner.

(2) Roughened surfaces reached higher strengths than plain surfaces. However, the type of surface preparation, that is, the degree of roughness, chipping, keying, sandblasting did not result in significant difference in strength.

(3) Filling up a gap between existing and new concrete with grout packed in the gap produced a very poor interface.

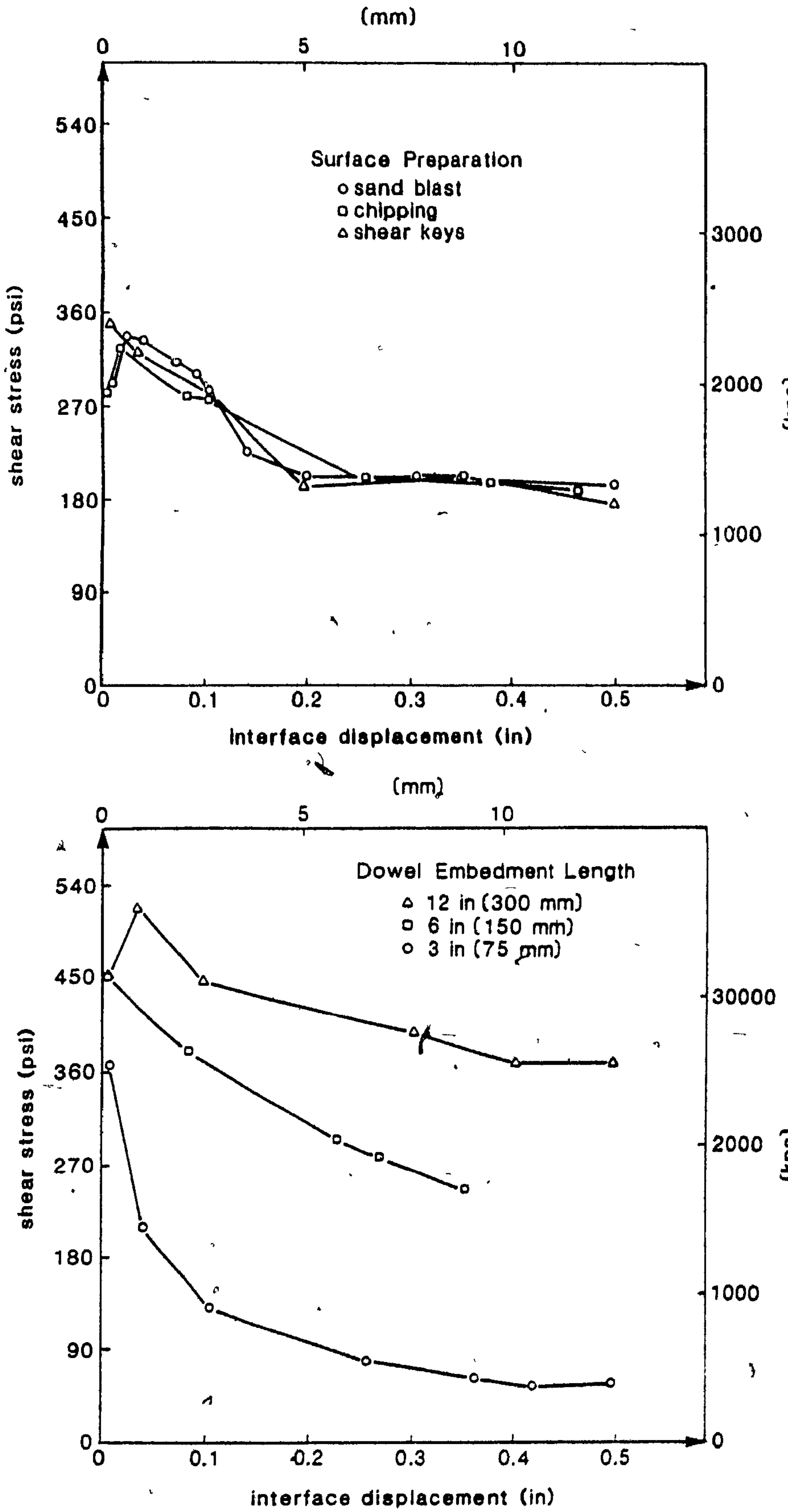


Figure 5.14 Envelopes of Shear Stress-Displacements Curves[31].

CHAPTER 6

CONCLUSIONS

The evaluation and upgrading of existing structures is becoming an increasingly important problem for Canadian engineers. Until very recently there has been very little guidance, in the form of codes or recommended design practice, for both the evaluation and the upgrading of different types of structures. The existing Canadian code as well as the recommendations of the Applied Technology Council and the Chinese code approaches were first reviewed. In addition the bylaws adopted by some municipalities were also reviewed.

It is felt that the current Canadian code often acts as a deterrent to the upgrading of structures because it requires full upgrading for structures being modified. In many instances no upgrading is carried out even though significant modifications are made to a structure. The Applied Technology Council approach considers the seismic risk and also permits partial upgrading for many structures. A tentative proposal for adapting the provisions of the Applied Technology Council to the Canadian context is presented. It is felt that this approach is well suited to the format of the National Building Code of Canada.

A brief review of some of the highly vulnerable types of construction is presented. Common deficiencies of different types of construction and possible methods of alleviating these deficiencies are summarized.

In order to provide guidance to engineers on possible methods of upgrading structures, a number of case studies of structures repaired and strengthened following the 1985 Mexican earthquake are reported. Although these methods of upgrading may be viewed as being too severe for Canada, they offer Canadian engineers valuable qualitative guidance. In addition, a number of case studies of Canadian structures that have been upgraded are also presented. These cases provide practical examples of both full upgrading measures for important structures as well as cases in which the engineer has sought permission from the local building official to provide partial upgrading.

A number of experimental studies, reported in the literature, on different techniques of upgrading concrete structures is presented. A theoretical study of the influence of different upgrading methods, taking into account the complex load history effects, is given.

It is hoped that the review of different approaches to the evaluation and upgrading of concrete structures as well as the case studies will provide some useful assistance to Canadian engineers faced with this difficult task.

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