ISTANBUL TECHNICAL UNIVERSITY * INSTITUTE OF SCIENCE AND TECHNOLOGY

STATE-OF-THE-ART IN REPAIR AND STRENGTHENING OF REINFORCED CONCRETE BUILDINGS

M. SC. THESIS

Ali Shoeb Abdulla, B.Sc.

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Supervisor : Prof. Dr. Halit DEMIR
Member : Prof. Dr. Zekai CELEP
Member : Assist.Prof. Dr. A. Necmettin GUNDUZ

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PREFACE

This study summarize the symposium about the methods and techniques used in assessment, repair/strengthening as well as materials for interventions on reinforced concrete buildings. Also summarize the reliable experimental data available on the repair/strengthening of reinforced concrete structural members. Some analytical and experimental studies are being carried out in various European, Asian, and North American Countries, especially Balkan Countries, Japan, and United State of America, which are related to repair and strengthening of concrete buildings. These experimental researches are concerned with the behavior and strength of repaired or strengthened building members and systems.

I must express my gratitude to my supervisor Prof. Dr. Halit Demir for his continuous support and encouragement over the years of writing this thesis, and I also acknowledge the help of the staff of civil engineering department. Special thanks are owed to my wife Zienab for her high limit state of patient and encouragement over the years which made the writing of this thesis a reality.

CONTENTS

NOTATION

The following symbols are used in this study:

a_{res} = residual deflection;
a_{tot} = total deflection;
A_o = total area of door or window;
A_t = total area of the compartment;
A_{si} = area of steel at fiber i;
A_t = area of tension reinforcement;
A_{t2} = additional steel area of tension reinforcement;
A'_c = cross-sectional area of additional concrete jacket;
A_s = area of steel plate, also area of tension reinforcement;
ΔA_c = area of added concrete;
A_c = area of existing concrete;
$A_{s,w}$ = area of new closed stirrups;
A_t = area of column longitudinal reinforcement at tension side;
A = total cross-section area of wall including boundary column;
A_w = area of horizontal reinforcement in wall;
A_g = total area of longitudinal reinforcement in boundary column;
a = h/3
A_{ds} = area of steel stud bolt;
A_f = area of flange of beam or column region;
A_c = effective area of concrete corn projected by a post-installed anchor;
A_e = minimum cross-sectional area of mechanical wedge anchor;
A_o = minimum cross-sectional area of spliced bolt or rebar;
${}_sA_e$ = cross-sectional area of post-installed anchor at interface;
b = width of column or beam;
b_e = A/L;
b_i = width of member at i fiber'
b_2 = width of compression face of column after strengthening;
C = basic shear-coefficient;
C = strength index;
d = effective depth of section, or diameter of bent down bars;
d_a = diameter of post-installed anchor, or nominal diameter of mechanical wedge anchor;
d_b = diameter of hole drilled;

d_{si} = depth of steel layer i;
d_{ci} = depth of concrete layer i;
dE = dissipated energy increment;
D = damage index;
D_a = diameter of wedge;
E_1 = young's modulus in direction 1;
E_2 = young's modulus in direction 2;
E_o = basic structural performance index;
E_s = modulus of elasticity ofv steel;
E_c = modulus of elasticity of concrete;
F = ductlity index, or yield point strength of steel;
f_c = compression strength of existing concrete;
f_{c1} = compression strength of new concrete;
f_{ct} = tensile strength of concrete;
f_{cc} = cube concrete strength;
$f_{ct,m}$ = tensile strength of concrete as it is monolithic;
f_{sy} = yield stress of steel;
f_{ck} = concrete cylinder strength;
f_{cr} = unit compression stress in the brace member;
$F_{s,d}$ = design force;
G = local geolgical index to modify the E_o-index;
G_g = shear modulus of the glue;
g = unsupported length of hoop side between legs of supplementray cross ties;
g_2 = unsupported length of hoop side between legs for additional hoop;
H = effective flexural length of column;
h = cross-section depth of column or beam;
h_2 = total cross-sectional depth of column after strengthening;
h_i = total depth of member at i fiber;
I = seismic performance index for structure;
j = 0.8d
K' = residual stiffness;
K = stiffness;
K_m = stiffness as if the final section were monolithic;
K_r = the real stiffness of the final section;
L = distance from extreme compression fiber to extreme tension fiber in wall;
l = anchor length, or total length of panel plate or bracing;
l_a = anchorage length, or entire length of mechanical wedge anchor;
l_e = effective anchorage length ;
l_d = total length of stud bolt or rebar which is spliced with post-installed anchor;

l_n = effectiv anchorage length of spliced steel bolt or rebar;

l_o = the half length of jacket;

l_s = shear ratio = l/d;

l_w = clear length of the in-filled wall;

l_{ai} = anchorage length;

l_1 = embedded length of mechanical wedge anchor;

l_2 = projected length of mechanical wedge anchor;

M =bending moment, or total mass of combustibles in the room;

M_{cr} = first cracking moment;

M_{st} = stabilized cracking moment;

M_o = moment corresponding to the elastic limit of steel;

M_u = ultimate bending moment, or ultimate flexural strength;

$\Delta M_{u,add}$= additional bending moment capacity;

$M_{u,exist}$= existing bending moment capacity;

$M_{u,m}$ = the bending moment strength as if the final section were monolithic;

$M_{u,r}$ = the real bending strength of the final section;

${}_wM_u$ = ultimate flexure strengtyh of wall with boundary columns;

${}_bM_y$ = flexural strength of beam region;

${}_cM_y$ = flexural strength of column region;

N = compressive axial force;

N_{tot} = total normal force, or total axial load to be retained after taking off temperary shores;

N_s = load to be transferred by means of additional reinforcement (A'_s) welded on existing reinforcement;

N_{res} = load to be trusted to the damage column;

N_f = load to be transferred to the section of the jacket (A_c') by means of lateral friction;

N_c = compressive strength;

N_t = tension strength;

N_1 = axial force by gravitational load at tension side;

N_2 = axial force by gravitational;

n_o = normalized axial stress = N/bdf_c';

p = normalized steel ratio = $p_t f_y/f_c'$;

p_c = edge column reinforcement ratio;

p_h = horizontal reinforcement ratio;

p_g = $A_g/b_e h$;

P_t = tension steel ratio,(and for wall

p_t =$100A_t/b_e.L$;

p_w = confinement ratio, or shear reinforcement ratio;

Q = ratio of fire load to unit area, or shear strength;

Q_a = shear capacity per a post-installec anchor;

Q_{a1} = shear capacity per a post-installed anchor that is dominated by yield strength of steel;

Q_{a2} = shear capacity per apost-installed anchor that is dominated by bearing strength of concrete;

Q_c = the smaller of flexural strength or shear strength of the other side of boundary column;

$_pQ_c$ = punching shear strength of the one side of boundary column;

Q_D = design shear force;

Q_{mu} = shear force of a column in which the yield hinge developed at both the top and bottom of a column;

Q_j = shear strength of post-installed anchor;

Q_{su} = ultimate shear strength of a column after strengthening;

Q_y = calculated yield strength;

$_wQ_{su}$ = ultimate shear strength of a wall after strengthening;

$_sQ_{su}$ = lateral load capacity of a frame after strengthening by steel bracing;

$_sQ_u$ = lateral load capacity that is dominated by steel bracing system, such as the strength of steel brace or steel panel;

q_{ds} = the unit strength of steel stud bolt;

R = strength;

R' = residual strength;

S = action-effect;

S' = new action-effect;

s = local slip, or spacing of stirrups;

s_u = critical slip;

sd = front slip;

T = time index to modify the E_o-index;

T_a = tension strength per a post-installed anchor;

T_{a1} = tension strength per a post-installed anchor, that is dominated by its yield strength;

T_{a2} = tension strength per a post-installed anhor, that is domnated by embedded concrete strength;

t = thickness of jacket;

t_c = tickness of concrete element;

t_g = thickness of glue;

t_s = thickness of steel plate;

t_w = thickness of in-filled wall;

V = shear force to be retained by the R.C. element after repair and/or strengthening;

$V_{u,r}$ = the real shear strength of the final section;

$V_{u,m}$ = the shear strength as if the final section were monolithic;

V_s = shear force which can be transferred by plate;

V_{exist} = shear force to be trusted to the existing (damage and/or weak) R.C element;
W = effective heat of combustion;
x = distance of netural axis from compression side;
α = E_s/E_c, or deformation compatibility factor;
α_h = end spacing of new stirrups used for repiaring damaged column;
β = coefficent for cyclic loading effect (function of structure parameter);
$d\gamma_{12}$ = incremental shear strain in the 1-2 plane;
γ_c = partial factor for concrete;
γ_m = safety factor for material;
γ_n = resistance-model concrete factor for redesign;
δ_m = maximum response deformation;
δ_u = ultimate deformation capacity under static load for beam and column;
δ = average settlement;
$d\epsilon_1$ = incremental linear strain in direction 1;
$d\epsilon_2$ = incremental linear strain in direction 2;
ϵ_{cr} = top strain of concrete;
ξ = rate of increase of residual deformation;
λ = ratio of residual to total deflection
λ = effective slenderness ratio;
μ' = residual ductility;
μ = ductility factor;
μ = friction coefficient corresponding to $\sigma = f_{ct,min}$;
v = poisson's ratio;
v_o = length along which densely spaced hoops have to be provided in concrete jacket for repair of damage column;
σ_c = compression stress of concrete;
$d\sigma_1$ = incremental normal stress in direction 1;
$d\sigma_2$ = incremental normal stress in direction 2;
σ_{si} = steel stress at i fiber;
σ_{ci} = concrete stress at i fiber;
σ_y = yield strength of longitudinal reinforcement;
σ_o = N/b.h;
σ_B = compression strength of existing concrete;
$_m\sigma_y$ = yield strength per a post-installed wedge anchor;
$d\tau_{12}$ = incremental shear stress in the 1-2 plane;
τ_b = bond stress;
τ_{α} = local bond strength;
τ_{α} = mean value of bond stress;
τ_o = basic bond strength of chemical adhesive;
Φ_h = diameter of bar used for hoop.

LIST OF FIGURES

SUMMARY

The purpose of this thesis to review some basic facts about concrete damage, and review some techniques used for interventions by repairing and/or strengthening, before and after building damage.

The necessary procedures for the inspection of damaged and/or weak building, and assessment of an existing (damage or not) buildings are presented in this document including temporary protection. Estimation of residual structural characteristics. Also quantifying damage and difinition of damage is breifly discussed.

Polymer materials are commonly used in the repair of concrete structures. These polymer materials include the epoxy resins, producing polymer concrete, and some non-polymer materials, such as special cement and additives are used in repair works. Traditional normal cast-in-place concrete is often used in repair and strengthening works, but always the results are unsatisfactory, mainly because of the volume change shown by the cement-based concrete used, that impairs every possibility of good contact between the new a nd the old element. The basic properties and recommendations to use these materials are reviewed in this study. Additional reinforcements, the use of incorporated steel profiles, also infilling and bracing techniques are briefly reviewed. Shotcrete (gunite), and grouts also discussed.

The techniques used for repair/strengthening of buildings, which have a back analytical and experimental data related to the different elements in a building (columns, beams, column-beam joints, slabs, shear walls, foundations) are summarized with some recommendations and provisionals. Also introduction of new structural elements (infilling shear walls, adding side walls to existing frames, infilling steel brace into existing frame, and adding buttresses to existing frame building) are summarized and discussed.

The calculation method used for dimensioning of additional structural arrangement for repair and strengthening for different elements in reinforce concrete buildings (columns, beams, in-filled shear walls, steel bracing) summarized in last chapter.

ÖZET

BETONARME BİNALARIN ONARIM VE GÜÇLENDİRŞLMESİNDE BUGÜNKÜ DURUM

Bu tez çalışmasının amacı, beton hasarı konusunda bazı temel hususlar ve yapı hasarından önce veya sonra onarım ve/veya yapıların güçlendirilmesi için kullanılan bazı tekniklerin gözden geçirilmesidir.

Hasar görmüş ve/veya zayıf olan yapıların akılcı projelendirilmesi mühendisin karşılaştığı en zor problemlerden biri olduğu kabul edilmektedir ve bunun belirli sayıda sebebi vardır; mevcut yapıların değerlendirilmesi ile ilgili belirsizlikler, alışılagelmişin dışında analitik modellere olan ihtiyaç ve ayrıca mühendisin yapı hakkındaki malumatının da genellikle çok mükemmel olmaması. Hasar görmüş bir yapının hasar seviyesi ve bu seviyeyi göstermek için kabul edilen yollar herhangi bir teknik yönetmelikte yeralmamıştır. Hasar görmüş ve zayıf yapıların iyileştirilmesine ait önemli adımlar Şekil 1 de görülmektedir. (Betonarme yapıların değerlendirilmesi ve yeniden projelendirilmesi için düşünülen model)

Analitik ve deneysel verilere sahip olan betonarme yapıların onarımı ve güçlendirilmesi ile ilgili bazı temel kavramlar kısaca gözden geçirilmiş ve bu alanda bugünkü durum ve bu maksat için kullanılan malzeme ve teknikler kısaca verilmiştir.

Onarım/güçlendirme pratiği araştırıcıların dikkatini sadece son zamanlarda çekmeye başladı, fakat onarım ve güçlendirme uygulama olarak yapım pratiği kadar eskidir. Yapım kalitesi oldukça zayıf olduğundan problemli durumlara çok sık rastlanılan gelişmekte olan ülkelerde onarım/güçlendirmeye ait iyileşmeler daima yaygın olarak mevcuttur. Dünya ülkelerinin çoğunda onarım ve güçlendirme uygulaması sezgiye dayanmaktadır. Yapısal elemanların iyileştirme sonrası davranışı ve sonuç olarak tüm yapı sisteminin davranışı üzerindeki

etkisi ne bilinir ne de bakılır.

Bu çalışma batonarme yapıların iyileştirilmesi için betonarme binaların değerlendirilmesinde kullanılan metod ve teknikleri biraraya getirmesi yanında onarım/güçlendirme için kullanılan malzemeleri de özetlemektedir. Ayrıca betonarme yapı elemanlarının onarım/güçlendirilmesi hakkında mevcut güvenilir deneysel verileri de özetlemektedir. Bazı analitik ve deneysel çalışmalar değişik Avrupa ülkelerinde, Asya ve Kuzey Amerika ülkelerinde, özellikle Balkan ülkeleri, Japonya ve Amerika Birleşik Devletleri'nde yürütülmektedir.

Hasar görmüş ve/veya zayıf binaların muayenesi için gerekli işlemler ve mevcut (hasarlı veya hasarsız) bir binanın değerlendirilmesi geçici koruma (Pathology image, geçici koruma için güvenlik tedbirleri, kalın yapısal karakteristliklerin tahmini) belgelerinin tahmini ile Bölüm 2 ve Bölüm 4 de sunulmuştur. Hasar görmüş ve/veya zayıf binanın gözden geçirilmesinde ana gaye, yapıdaki hasarın artması durumunda muhtemel malzeme kayıplarını en aza indirmeye kadar yapıdaki hasarın nitelik ve derecesini ayrıntılarıyla belirlemek ve kaza ve yaralanma riskini ortadan kaldırmak için geçici mesnetlerle güvenlik tedbirlerini yerleştirmektir. Kusurlu veye hasar görmüş betonarme bir yapının yeniden projelendirilmesinde önemli hazırlayıcı adımlar, ölçüm ve testler yanında yapı hakkında verilerin toplanması ve yapının muayenesidir. (Gözden geçirilmesi, incelenmesi) Bu adımlar kazai etkilerin bilinmediği zaman bunların tahmin edilmesiyle Bölüm 2' de kısaca tartışılmıştır.

Geçici koruma için önerilmiş olan teknikler düşey ve yanal mesnet oluşturma metotlarının dahil edilmesiyle Bölüm 3'de tartışılmıştır. Geçici koruma tedbirlerinin hasar görmüş binaya yakın oturan alanlardaki insanlara emniyet sağladığı,ayrıca tamir ve güçlendirme yapan işçilere güvenlik sağladığı, bu geçici koruma tedbirleri bazı temel işlemlerle kısaca incelenmiştir.

Kalıcı yapısal karakteristliklerinin hesabı (rijitlik, mukavemet,düktilite) Bölüm 4 de genel analitik metodlarla,mekanik etkiler olması durumunda malzemenin kuramsal kanunları,yangın ve korozyon sonrasının dahil edilmesiyle kısaca gözden geçirimiştir. Farklı yükleme şartları altında (monoton gerilme davranışı, tekrarlı gerilme davranışı, tekrarlı geri devirli gerilme davranışı) çeliğin davranışı yukarıda bahsedilen davranışları yansıtması için önerilen çeşitli analitik metodların incelenmesiyle Bölüm 4 de kısaca özetlenmiştir. Son 20 senede önerilen betonun geri devirli yükleme altında bir, iki ve çok eksenli davranışının yaklaşımı için çok sayıda malzeme modelleri de Bölüm 4 de kısaca verilmiştir. Monotonik ve geri devirli etkiler altında yerel aderans-yerel kayma modelleri de özetlenmiştir. Ayrıca farklı araştırıcılar tarafından rapor edilen betonarme yapıların çelik ve beton bileşenleri üzerine yangın etkileri de Bölüm 4 de özetlenmiş bulunmaktadır.

Eğilme ve eksenel yük etkisindeki betonarme elemanlerın kalıcı yapısal karakteristlikleri malzemenin kuramsal kanunlarına dayanılarak, moment-eğrilik didiyagramlarının hesaplanmasıyla elde edilebilir. Moment-eğrilik ilişkisinin hesabında kullanılan metotlar analitik bir düzlem olan genel lif metodu kısaca özetlenmiştir. Model

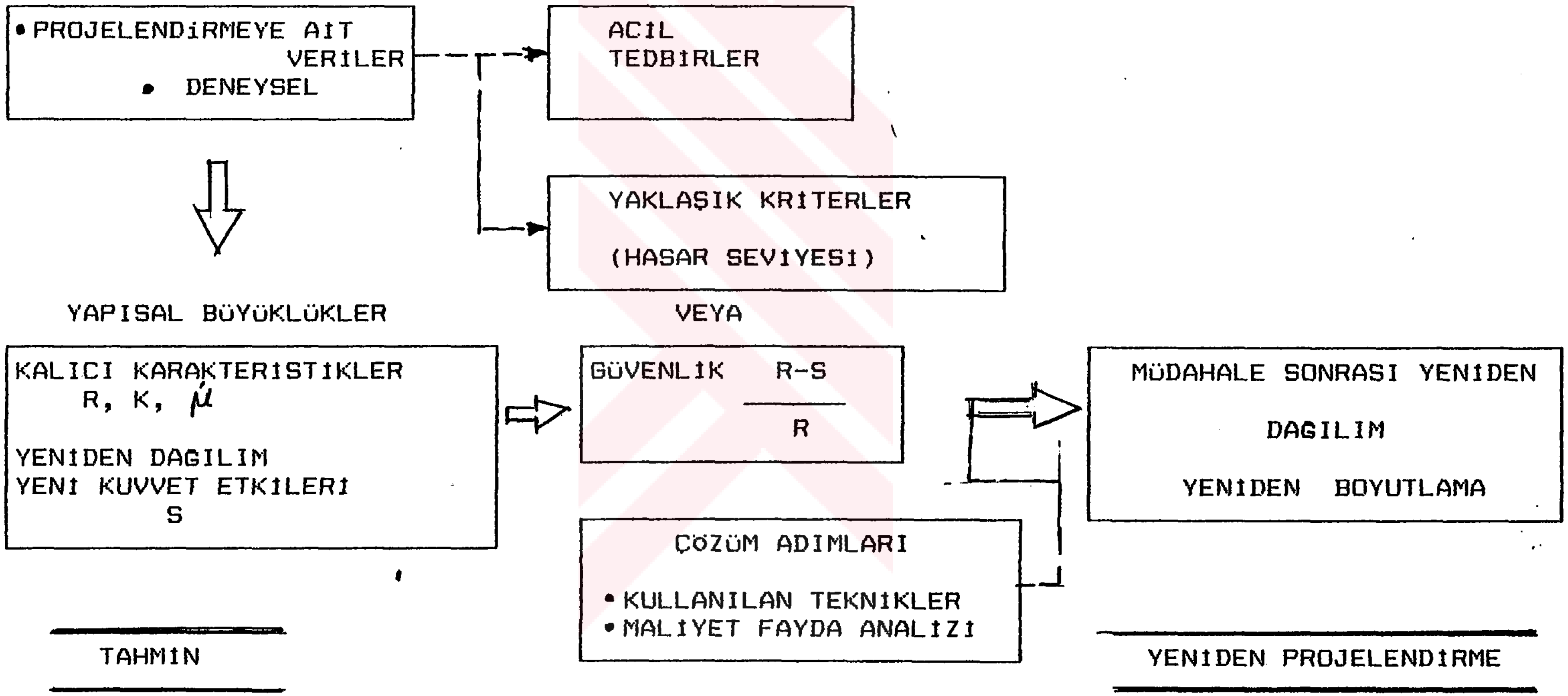
Akılcı bir yolla hasar belirlemek için metod bulmaya yönelik girişimler hasarın tam ve hassas tanımıyla başlamalıdır. Çünkü hasar kelimesi yaygın olarak farklı bütün olaylarda kullanılmakta ve kişiye göre farklı tercüme edilmektedir. Betonarme elemanların hasarı, ilgili elemanların daha fazla yük taşıma kapasitesini netice veren fiziksel bozulmanın özel bir derecesi olarak belirtilmelidir. Benzer olarak bir elemanın göçmesi, kişiye göre farklı olarak tanımlanan kalıcı yük taşıma kapasitesinin belirlendiği özel bir hasar seviyesini belirtir. Betonarme bir elemanın hasarının yukarıdaki tanımı, hasar mefhumu, betonarme elemanların hasar ve göçmesinin temel tartışması ve beton hasar modelleri Bölüm 5 de kısaca özetlenmiştir.

Onarım ve/veya güçlendirme işlerinde çeşitli teknolojiler ve malzemeler yaygın olarak kullanılmaktadır. Proje yapanlar bu malzeme veya tekniklerin bazılarında tecrübesiz olabilirler. Betonarme yapıların onarımında polimer malzemeler yaygın olarak kullanılmaktadır. Bu polimer malzemeler epoksi reçineleri, oluşturulan polimer betonu ve bazı polimer içermeyen özel çimento ve katkıları gibi malzemeler onarım işlerinde kullanılmaktadır. Geleneksel olan normal yerinde döküm beton onarım ve güçlendirme işlerinde sık sık kullanılmaktadır, fakat kullanılan betonun çimentoya bağlı olarak hacim değişikliği yapması dolayısıyla eski ve yeni eleman arasındaki iyi temasın bozulması ihtimaliyle sonuç daima memnuniyet verici değildir. Bu malzemeleri kullanmak için temel özellikler ve öneriler Bölüm 6 da gözden geçirilmiştir. Püskürtme beton ve harçlar da Bölüm 6 da tartışılmıştır. Onarım ve/veya güçlendirmede kullanılan malzeme ve tekniklerin özetlenmesinin amacı, bir yapı mühendisi bir onarım/güçlendirme projesini ele aldığında evvela kullanılacak olan onarım malzemesi ve gerekli uygulama özelliklerine karar vermesi gerektitiği içindir.

Onarım/güçlendirme de kullanılan tekniklerle ilgili pek çok temel kavramlar deneysel çalışmalarla doğrulanmıştır. Yapısal anlamda onarım,hasarlı eleman veya yapının yapısal dayanımını eski durumuna getirme işlemi, ve güçlendirme ise yapı elemanı veya yapının dayanımının eski durumunun üzerinde bir düzeye getirme işlemi olarak tanımlanabilir. Betonarme binaların onarım/güçlendirilmesine ilişkin mevcut güvenilir deneysel veriler ve kullanılan teknikler Bölüm 7 de tartışılmaktadır. Yeni yapısal elemanlara (perde duvarları, mevcut çerçevelere duvar ilavesi, mevcut çerçevelere çelik çarprazlar ve mevcut çerçeve binalara payandalar) giriş de özetlenmiş ve aynı bölümde tartışılmıştır.

Son iki bölüm projelendirme işlemi ve hesap teknikleri ve ilave düzenlemelerin boyutu ile ilgilidir. Betonarme binalarda farklı elemanlar (kolonlar, kirişler, ilave perde duvarlar, çelik çaprazlama) da onarım ve güçlendirme için ilave yapısal düzenlemede kullanılan hesap metodu son bölümde özetlenmiştir.

Hasar görmüş ve/veya zayıf betonarme yapıların projelendirilmesi ve değerlendirilmesinde kullanılan temel teknik ve metodları kapsayan bu çalışma, betonarme binaların yeniden projelendirilmesi ve değerlendirilmesinde konu ile ilgili mimar ve mühendislere rehber olacaktır.



Şekil 1. Betonarme Yapıların İncelenmesi ve Yeniden Projelendirmesi İçin Düşünülen Model
R=Mukavemet, S
K=rijitlik, μ :Süreklik

Chapter 1

INTRODUCTION

Worldwide the investigation and evaluation of existing structures, and where necessary their strengthening, has been carried on for many years with the resultant accumulation of considerable theoretical and practical experience. However, from the literature, it is clear that there is no unanimity of opinion among engineers as to the proper methodology for strengthening an existing concrete building. The investigation, evaluation and determination of whether strengthening is required or not, and how to strengthen, is still as much an art as it is a science. Codification of repair and/or strengthening procedures can never remove the necessity for art in the application of such procedures.

The concept of damage permeates many branches of concrete engineering. As concrete is subjected to loads of increasing intensity, it undergos different phases of damage, from microcracking up to ultimate failure.

Any attempt to devise methods to quantify damage in rational way should begin with a clear and precise definition of damage because "damage" is a widely used word that describes all kinds of different phenomena and is open to subjective interpretation. Damage of reinforced concrete member shall signify a specific degree of physical deterioration with precisely defined consequences regarding the members capacity to resist further load. Similarly, "failure" of a member signifies a specific level of damage, which corresponds to a somewhat arbitrarily defined residual capacity to resist further load.

Experience from recent earthquakes has demonstrated that structures which have been properly designed and constructed are able to withstand severe earthquakes without collapse. However, these earthquakes have shown that old buildings as well as building of recent construction can be seriously damaged or can collapse causing loss of life to the occupants. Studies of the structural performance have clearly demonstrated that structural system must not only have sufficient strength to resist any type of forces, but they also must have sufficient ductility, or the ability to maintain their integrity when stressed beyond their yield point in order to protect human life.[1]

Among several classification of damages, it is interesting for this document to recall a classification regarding damaging action only, (i.e a classification which does not deal with the use of the building, the stage of its construction, ect..).[2]

a) Exceptional and accidental actions:
   1. earthquake, landslides
   2. hurricanes, tornadoes
   3. explosions (detonations, container explosions, etc.)

4. impacts (from vehicles, ships, air crafts, etc.)
5. incidents due to prestressing operations

b) Aging-environmental actions:
   1. chemical attack (corrosion of steel and/or concrete)
   2. carbonation
   3. freezing-thawing

c) Imposed deformation
   1. differential settlements
   2. thermal-effects
   3. time effects (creep, shrinkage)

d) Fire

e) Overloading (vertical loads)

f) Construction gross-errors
   1. inadequate concrete quality
   2. inappropriate geometry
   3. insufficient reinforcement or detailing

For each one of these actions, some particular effects have to be examined:[2]

i) possible modification of constitutive laws of materials and/or of critical regions of building elements

2. particularities in facing assessment and redesign

Repair is the reestablishment of the initial strength of damaged structural members and the reestablishment of the function of damaged nonstructural elements. Properly repaired structural members will possess approximately the same strength as before they were damaged but will probably have a somewhat reduced stiffness in concrete and masonry structures due to very fine cracks which are caused by the earthquake and are impossible to restore.

Strengthening is the judicious modification of the strength and/or stiffness of structural members or of the structural system to improve the structure's performance in future earthquakes or other causes. Strengthening generally includes increasing the strength or ductility of individual members or introducing new structural elements to significantly increase the lateral force resistance of the structure.

Each structure is a unique system and its damage will be different from other structures, requiring custom repair and/or strengthening solutions and details. The decision of the repair and/or strengthening method and the appropriate construction technique depends on many factors, such as local site conditions, type and age of the structure, type and degree of damage, available time, equipment and staff for specific repair and/or strengthening work, architectural requirements, cost, and the required level of safety.

The key steps of interventions on damage and/or weak R.C structures as shown in (fig. 1.1) are:[2]

1- Urgent measures, such as demolition, evacuation, relief of loads, propping, shoring, bracing, etc.
2- Assessment and diagnosis (informational and experimental data on residual structural characteristics).

3- Remedial concepts (urgency, available personnel and equipment, available techniques), including cost-benefit considerations, social, historial and other considerations.

4- Redesign procedures (conceptual redesign, structural analysis, estimation of action-effects, redimensioning of structural arrangement).

The necessary procedures for the assessment of an existing (damaged or not) structure are presented in chapter 2 through chapter 5 of this document including temporary protection ("investigation and evaluation, emergency measures for temporary protection, estimation of residual structural characteristics, damage of R.C building"). In chapter 6 and chapter 7 the materials and techniques used in repair and/or strengthening are discussed and reviewed. Chapter 8 and chapter 9, design procedures for repair and strengthening of R.C building combined with appropriate criteria and redimensioning with calculation techniques used in redesign procedures are presented.

In connection a glossary should be reminded very briefly:

1. Assessment: evaluation of the condition and the mechanical characteristics of the structure
2. Interventions: repair, substitution, strengthening
3. Repair: restoring of the initial mechanical characteristics (reinstatement)

4. Substitution: demolition and rebuilding of (heavily) damaged elements
5. Strengthening: instating of mechanical characteristics higher than the initial ones
6. Redesign: design procedures concerning interventions

Rational design of existing damaged and/or weak structures has not known the same degree of refinement such as design of new structures, and there is a certain number of reasons for it: Uncertainties related to the assessment of existing structures, as well as the unconventional analytical models needed (taking into account several discontinuities), are but few of the difficulties encountered in "redesign" not to mention the need for a more general and complex reliability philosophy.[2]

This thesis is therefore an attempt to be a State-of-the-Art Report, containing a first (very brief though) synthesis of knowledge on the subject, able to serve as a practical guide for a more rational redesign of repaired and/or strengthened concrete building structures.

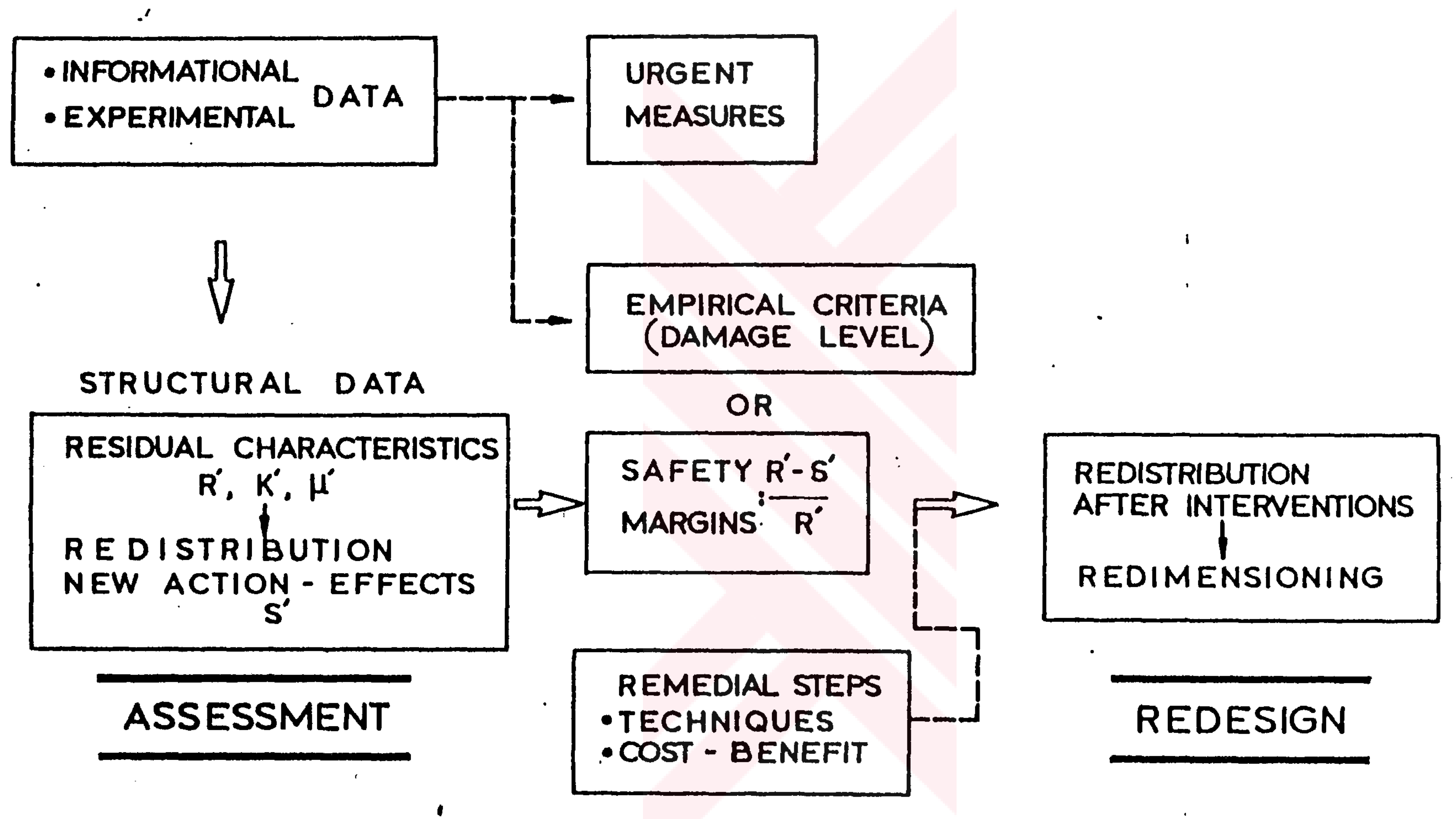


Fig. 1.1 A thinking model for assesssment and redesign of R.C structures
R: strength, S: action-effect
K: stiffness, μ: ductility

Chapter 2

INVESTIGATION AND EVALUATION

The engineer concerned with the problem of a concrete structures, is facing first of all the problem of the assessment of the present condition of the structure. The point to be made is that the engineer charged with or interested in assessment must be trained , technically, in where to look, how to look, and what to look for, before he can even be expected to realize that there is trouble, knowing what, where, and how to look also requires a knowledge of the various types of damages which can occur and of the basic causes of damages, and experience.

This assessment may be based on either a group of informational and experimental data (inspection, measurements, tests, etc.), in conjuction with an analysis based on current design method, or on overall loading tests (vertical as well as horizontal ones). In some cases these two groups of data may lead to somehow contradictory conclusions; therefore emphasis is given on the need for calibration and good engineering judgement.

2.1 Inspection, Measurements and Tests:

The main preparatory steps of the redesign of a defective or damaged R.C structure are, inspection and collection of informational data, as well as measurements and tests.[2]

2.1.1 Inspection of Damaged and/or Weak Buildings:

The main purpose of inspection of damage and/or weak building is to determine in detail the nature and degree of damage and to design and install emergency measures for temporary support to avert the risk of casualties and injuries, as well as to minimize the possible material losses in case of increased damage to the structure. The inspection of a damage building comprises the following stages:

a) Visual inspection: Visual examination and estimation of the consequences of the damage need to know three basic visual symptoms of distress in a concrete structure which are, cracking, spalling, and disintegration (which may be defined as a general decay of the surface involving loss of the cement past and loosing of the particles of coarse aggregate). Each of these basic symptoms in it self is fairly obvious and be readily detected and differentiated from the others.

Severe damage may appear in the form of a rupture of a column, or serious cracking of load bearing walls, etc. Possible urgent measures, such as evacuation of the building, propping and shoring, relief of loads, etc. are needed in this case.

b) Collection of information: Investigate the history of the structure. When was it built? By whom? In what season of the year? What type of cement was used? What kind of aggregates were used, and what was their source? What were the methods of construction? How was the concrete cured? Collection of information regarding the previous condition of the building, such as possible previous repair and/or strengthening work, behavior of the building during previous damaging action, pre-existing damage, etc. These data should also include the data of construction, and information regarding the use of the building, as well as the file of quality control, if any, during construction.

c) <u>Sketching (on plans and elevations)</u>: The location and of cracks should be noted on a sketch of the structure. a grid marked on the surface of the surface of the structure can be useful to accurately locate cracks on the sketch. Locations of observed spalling, exposed reinforcement, surface deterioration, and rust staining should be noted on the sketch. Sketching (on plans and elevations) of all kinds of damage of load-bearing as well as secondary elements (on existing or new drawings). photographs of damaged elements, which provide an overall view of the level and character of damage occurred.

d) <u>Localization of possible gross-errors, ignorance or carelessness</u>:

1- In the conceptual design of the building (especially as far as the earthquake and/or wind resistance design is concerned, in case of damages due to earthquake, hurricanes or tornadoes). The effects of improper design range from poor appearance to lack of serviceability to catastrophic failure.

2- In the construction and detailing can result in cracking in concrete structure because of poor construction and detailing.

3- In the maintenance and possible misuse

e) Study of design documents of the building:

Which includes the examination of construction drawing in order to evaluate the correctness of detailing foreseen, proof calculations in order to disclose possible errors regarding action-effects and dimensioning, especially in the damaged elements of the building, etc. If the initial design documents (drawings, proof calculations, etc.) are missing, new ones must be prepared. New drawings will show the general layout, dimensions and cross-sections of all members and the reinforcements in certain crucial regions. Also, new proof calculations have to be carried out, following the results of a new (brief but careful) structural analysis of the structure.

The above mentioned steps of inspection can be performed based on uniformly methodology used by " Balkan countries"[3] **Fig. 2-1** contains methodology for inspection of earthquake damaged and usability classification which can be generalized for inspection of any damaged and/or weak building.

SKETCH OF THE BUILDING
PLAN CROSS SECTION

ADDRESS:
OWNER:

1.TOWN (NAME) 1 ☐☐☐☐ 5

2. BUILDING IDENTIFICATION:
2.1 SECTION NUMBER OF CONSIDERED AREA OR SETTLEMENT: 6 ☐☐ 7
2.2 WORKING TEAM NUMBER 8 ☐☐ 9
2.3 NUMBER OF BUILDING 10 ☐☐☐ 12

3. ORIENTATION OF THE BUILDING PRINCIPAL AXIS (X): 1.NS, 2.EW, 3.N45E, 4.N45W 13 ☐

4. POSITION OF THE BUILDING IN THE BLOCK: 1.CORNER, 2.MIDDLE, 3.FREE 14 ☐

5. NUMBER OF STORIES:
5.1 STORIES 15 ☐☐ 16
5.2 APPENDAGES 17 ☐
5.3 MEZZANINES 18 ☐
5.4 BASEMENTS 19 ☐

6. GROSS AREA OF THE BUILDING (m^2): 20 ☐☐☐ 22

7. USAGE (SEE DESCRIPTION ON BACK PAGE):
7.1 BUILDING: 23 ☐☐☐ 25
7.2 GROUND FLOOR: 26 ☐☐☐ 28

8. NUMBER OF APARTMENTS: 29 ☐☐ 30

9. CONSTRUCTION PERIOD (TO BE DEFINED BY EACH COUNTRY): 1. 2. 3. 31 ☐

---

10. TYPE OF STRUCTURE (SEE DESCRIPTION BACK PAGE): 32 ☐☐☐ 34

11. FLOORS: .R.C., 2.STEEL, 3.WOOD, 4.OTHER 35 ☐

12. ROOF: 1.R.C., 2.STEEL, 3.WOOD, 4.OTHER 36 ☐

13. ROOF COVERING: 1.TILES, 2.METAL SHEETS, 3.LIGHTWEIGHT ASBESTOS CEMENT, 4.ASPHALT PAPER,5.HEAVY INSULATION, 6.LIGHT INSULATION, 7.OTHER 37 ☐

14. QUALITY OF WORKMANSHIP: 1.GOOD, 2.AVERAGE, 3.POOR 38 ☐

15. TYPE OF LOAD SYSTEM (SEE DESCRIPTION ON BACK PAGE): 1.BEARING WALL, 2.FRAMES, 3.FRAMES WITH INFILL WALLS, 4.FRAME WITH SHEAR WALL, 5.SKELETON WITH INFILL WALLS, 6.MIXED, 7.OTHER (SPECIFY) 39 ☐

16. FIRST FLOOR STIFFNESS RELATIVE TO OTHERS: 1.LARGER, 2.ABOUT EQUAL, 3.SMALLER 40 ☐

17. REPAIRS FROM PREVIOUS EARTHQUAKES: 1.NO, 2.YES, 3.UNKNOWN 41 ☐

---

18. DEGREE OF DAMAGE
STRUCTURAL ELEMENTS (SEE DESCRIPTION ON BACK PAGE):
18.1 BEARING WALL: 42 ☐
18.2 COLUMNS: 43 ☐
18.3 BEAMS: 44 ☐
18.4 FRAME JOINTS: 45 ☐

NONSTRUCTURAL ELEMENTS AND INSTALLATIONS (SEE DESCRIPTION IN THE MANUAL):
1.NONE, 2.SLIGHT, 3.MODERATE, 4.HEAVY, 5.SEVERE
18.11. INTERIOR WALLS: 50 ☐
18.12. PARTITIONS: 51 ☐
18.13. EXTERIOR WALLS (FACADE): 52 ☐
18.14. ELECTRICAL INSTALLATIONS:

Fig. 2-1 Form of methodology used in Balkan countries for inspection of earthquake damage

18.5 SHEAR WALLS: 46☐

18.6 STAIRS: 47☐

18.7 FLOORS: 48☐

18.8 ROOF: 49☐

53☐

18.15. PLUMBING: 54☐

---

19. DAMAGE OF ENTIRE BUILDING:
    1.1 NONE, 1.2.SLIGHT,
    2.1.MODERATE, 2.2.HEAVY,
    3.1.SEVERE, 3.2.TOTAL 55☐
20. INDIRECT DAMAGE (FIRE,SLAMMING.ETC.
    1.NO, 2.YES 56☐

21. OBSERVED SOIL INSTABILITIES AND GEOLOGICAL PROBLEMS
    1.NONE, 2.SLIGHT SETTLEMENTS, 3.INTENSIVE SETTLEMENTS, 4.LIQUEFACTION, 5.LANDSLIDE, 6.ROCKFALLS, 7.FAULTING, 8.OTHER(SPECIFY) 57☐

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22. USABILITY CLASSIFICATION AND POSTING:
    POSTED: 1.GREEN. 2.YELLOW, 3.RED
    NOT POSTED: 4. TO BE POSTED GREEN AFTER REMOVAL OF LOCAL HAZARD, 5.SOIL AND GEOLOGICAL PROBLEMS, REINSPECTION REQUIRED 6.UNABLE TO CLASSIFY, REINSPECTION NECESSARY,
    7. BUILDING INACCESSIBLE 58☐

GREEN	1	ORIGINAL SEISMIC CAPACITY HAS NOT BEEN DECRESED	UNLIMITED USAGE
YELLOW	2	ORIGINAL SEISMIC CAPACITY HAS BEEN DECREASED	TEMPORARILY UNUSABLE LIMITED ENTRY
RED	3	BUILDING DANGEROUS AS SUBJECT TO SUDDEN COLLAPSE	ENTRY PROHIBITED

MAIN REASONS FOR YOUR CLASSIFICATION AND POSTING:

23. RECOMMENDATIONS FOR EMERGENCY MEASURES:

    1.NONE, 2.REMOVE LOCAL HAZARD,
    3.PROJECT BUILDING FROM FAILURE,
    4.PROJECT STREETS OR NEIGHBOURING BUILDINGS, 5.URGENT DEMOLITION 59☐

24. ADDITIONAL DATA (PHOTO/SKETCHES AND COMMENTS): 1.NONE, 2.PHOTOS ONLY, 3.SKETCH AND COMM. ONLY, 4.PHOTOS AND SKETCH AND COMM. 60☐

25. ESTIMATED PRESENT VALUE OF BUILDING
    (MILLONS OF..........) 61☐☐☐63
26. ESTIMATED LOSSES (% OF ESTIMATED VALUE) 64☐☐☐66
27. HUMAN LOSSES (DEATHS AND INJURIES)
    (1)NO; (2)POSSIBLY; (3)YES
    (4)IF INFORMATION AVAILABLE PLEASE INDICATE 67☐

    NO. OF DEATHS 68☐☐69

    OPTIONAL NO. OF INJURIES 70☐☐71

28.DATE OF INSPECTION:MONTH/DAY
    NAMES OF INSPECTION ENGINEERS: 72☐☐☐☐75
    1. SIGNATURES
    2.
    3.

Fig. 2-1 (Continued)

7. BUILDING USAGE/IMPORTANCE CATEGORIES:
USAGE
10 Residential: 11 Family house,12 Apartment Buildings
20 Office: 21 Entire Building, 22 Partially
30 Economical: 31 Trade, 32 Finance, 33 Small industry, 34 Storage and Warehouses, 35 Agricultural,Fishing Forestry
40 Health and Social Welfare: 41 Hospitals and clinics, 42 Social welfare (old people home, invalid daycare canters)
50 Public Services: 51 Administrative-central or local government, 52 Police, 53 Fire station, 54 Transportation (buildings: ground,rail,air,sea) 55 Communications (building: postal, radio,TV)
60 Education and Culture: 61 Education, 62 Historical and religious.63 Cultural and entertainment, 64 Sports (gymnasium, stadium)
70 Tourism and Catering: 71 Hotels, 72 Restaurants,Cafe, 73 Coffee shops, pastry shops, etc.
80 Industry and Energy: 81 Industrial, 82 Energy (power Plant,transformer station, etc.)
90 Other Buildings (to be described)

10. TYPE OF STRUCTURE:

1st Digit	2nd Digit	3rd Digit
1= Masonry	1= No belts	1= Adobe
	2= Horizontal belts	2= Stone with no mortar
	3=Horizontal & Vertical belts, or Diagonal braces	3= Stone with mortar
		4= Solid brick
		5= Hollow brick
		6= Concrete blocks
	4= R.C. floors or roof	7= Unreinforced concrete
		8= ________
		9=

18. DEGREE OF DAMAGE:(damage category)
1. NONE: Without visible damage to the structural elements. possible fine cracks in the wall and celling mortar. Hardly visible nonstructureal and structural damage.
2. SLGHT: Cracks to the wall and celling mortar. Falling of large patches of mortar from wall celling surface. Considerable cracks, or partial failure of chimneys attics and gable walls.Disturbance, partial sliding, sliding and falling down of roof covering. Small cracks in structural members judged not to reduce, the seismic capacity of the building.
3. MODERATE: Diagonal or other cracks to structural walls, walls between windows and similar structural elements. Large cracks to reinforced concrete structural members: columns, eams, R.C.walls. Partially failed or failed chimneys, attics or gable walls. Disturbance, sliding and failing down of roof covering.
4. HEAVY: Large cracks with or without detachment of walls with crushing of materials.Large cracks with crushed material of walls between windows and similar elements of structural walls. Large cracks with small dislocation of R.C. structural elements: columns, neams and R.C. walls. definite dislocation of both structural elements and entire building.
5. SEVERE: Structural members and their connections are extremely damaged and dislocated. A lage number of crushed structural elements. Cconsiderable dislocations of the entire building and settlement of roof structure. Partially or completely failed building.

22. USABILITY RELATED TO POSTING (JUDGMENTAL)

GREEN - Buildings posted as green (damage category 1&2) are without decreased seismic capacity and do not appear to pose danger to human life. Immediately usable, entry unlimited.

Fig. 2-1 (Continued)

2= Reinforced Concrete	1= Cast in place frame	1= With lightweight partitions
	2= Cast in place bearing walls	2= With solid brick infills
	3= Prefabricated	3= With hollow brick infills
	4= Mixed with masonry	4= With concrete block infills
	5= Mixed with steel	
3= Steel	1= Heavy steel structure	1= With lightweight partitions
	2= Light steel structure	2= With solid brick infills
	3= Mixed with masonry or concrete	3= With hollow brick infills
4= Wood	1= Wood frame	1= ________________
	2= Bagdadi	2=________________
	3=Other	

<u>YELLOW</u> -Building posted as yellow (damage category 3&4) have significantly decreased seismic capacity.
Limited entry is permitted, but not usage on a continouos basis before repair and/or strengthening. Need for supporting and protection of the building and its surroundings should be considered.

<u>RED</u> - Building posted as red (damage category 5) are unsafe and subject to sudden collapse. Entry is prohibited. Protection of streets and neighbouring buildings or urgent demolitionmay be required. In case of isolated or standard building decision for demolition should be based on economical study for repair and strengthening.

15. TYPE OF STRUCTURAL SYSTEM:

1. Vertical and lateral loads are carried by bearing walls.
2. Vertical and lateral loads are carried by frames
3. Vertical and lateral loads are carried by frame and infills
4. Vertical and lateral loads are carried by frame-shear wall system
5. Vertical and lateral loads are carried by columns and walls but no well-defined frames are present.
6. Vertical and lateral loads are carried by combination of walls, frames, infills and/or shear walls.
7. Other systems to be described (e.g.inverted pendulum types, etc.)

Fig. 2-1 (Continued)

The Engineer concerned with such a problem of redesign of an "old" concrete structure should also take into account the standards (national Codes, design rules, practice, etc.) in force when the structure was constructed.

Finally, it has to be noted that among the above mentioned steps of inspection, an examination of other building similar in structural form in the vicinity should also be carried out, for purposes of differential diagnosis, especially in case of damages due to earthquakes, aging-environmental actions, differential settlements, etc.

2.1.2 Measurements and Tests:

Rudimentary techniques of structural evaluation by vibration monitoring are thousands of years old. Sounding of clay pots can reveal cracks, and tapping along the surface of a material can reveal voids. In many cases, instrumental measurements and tests may be needed, both in order to quantify the level and character of damage and to complete the information regarding the condition of the building before any repair and/or strengthening work.

a) Damage quantification:

1- Geometrical measurements:

(i) plumbing, leveling, eccentricities,

(ii) widths of cracks, crack width can be measured with an accuracy of about (0.025mm) using a crack comparator **Fig. 2.2** or similar instrument

(iii) residual deflections

(iv) in-time evolution of the above mentioned characteristics, possible installation of monitoring sensors, for example the crack movement can be monitored with mechanical movement indicators of the types shown in **Fig. 2.3**. The indicators, or crack monitor, in **Fig. 2.3**(a) gives a direct reading of crack displacement and rotation. The indicator in **Fig. 2.3**(b) amplifies the crack movement (in this case, 50 times) and indicates the maximum range of movement occurring during the measurement period.

2- Overall tests:

see subsection 2.2 (Criteria for Evaluation of Overall Loading Tests).

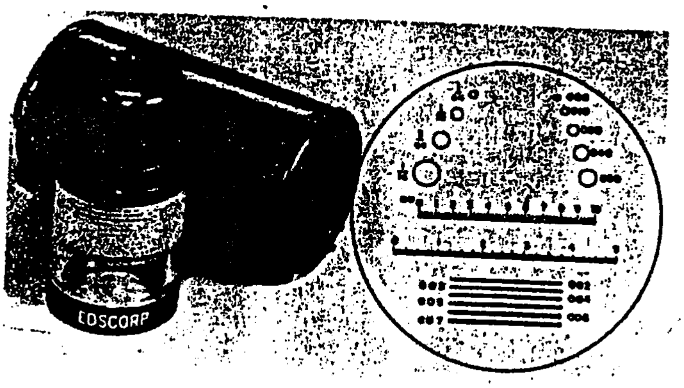
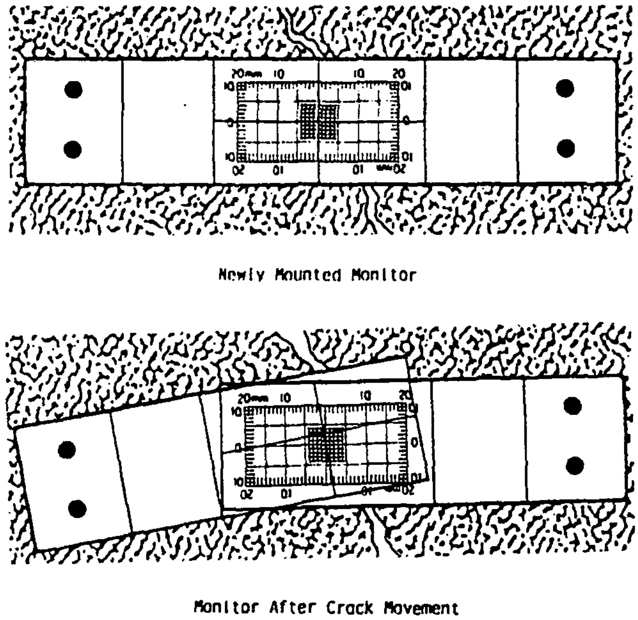
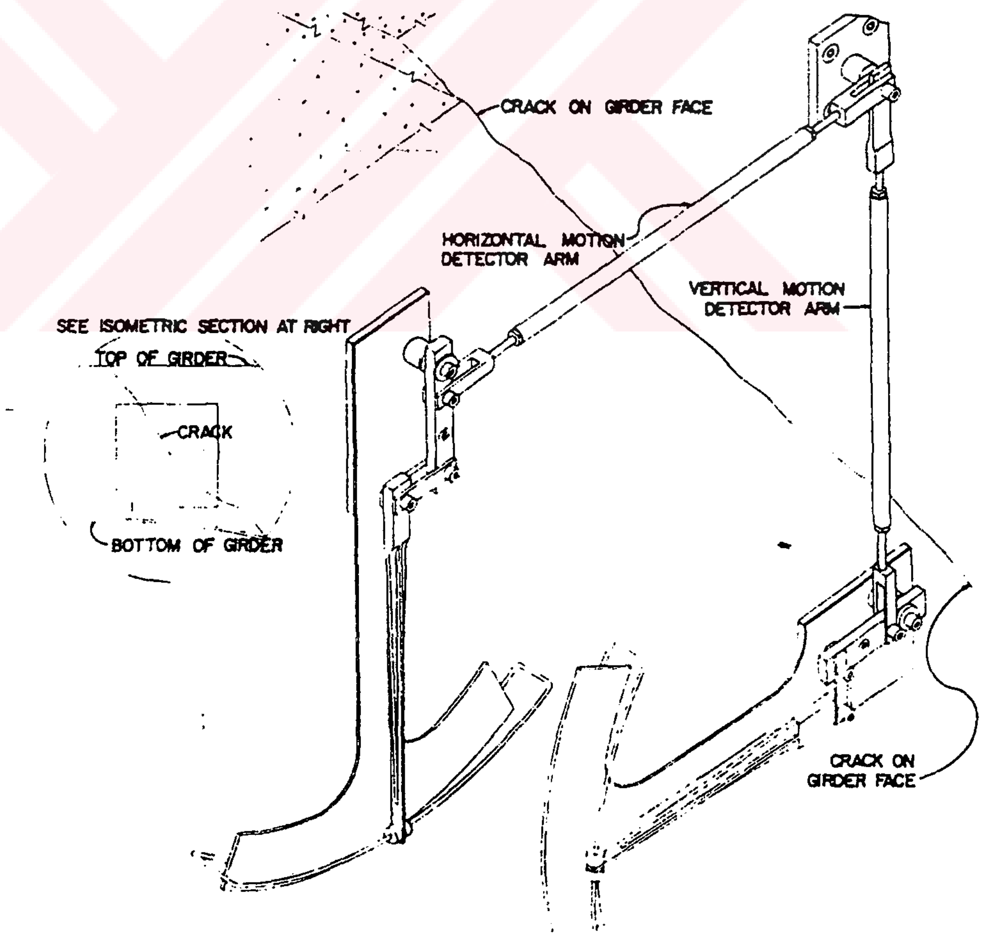


Fig. 2.2—Comparator for measuring crack widths



(a)—Crack monitor (courtesy of Avongard)



(b)—Crack movement indicator (Reference 34)

Fig. 2.3-Crack monitor and crack movement indicator

b) <u>Measurements and tests regarding previous condition of building</u>:

1- General arrangement of load-bearing and secondary elements, dimensions of cross-sections, uncovering and localization of reinforcements (the presence of reinforcement can be determined using a pachometer (Fig. 2.4). a number of pachometers are available that range in capability from merely indicating the presence of steel to those that may be calibrated to give either the depth or the size (provided that the other is known) of the reinforcing.

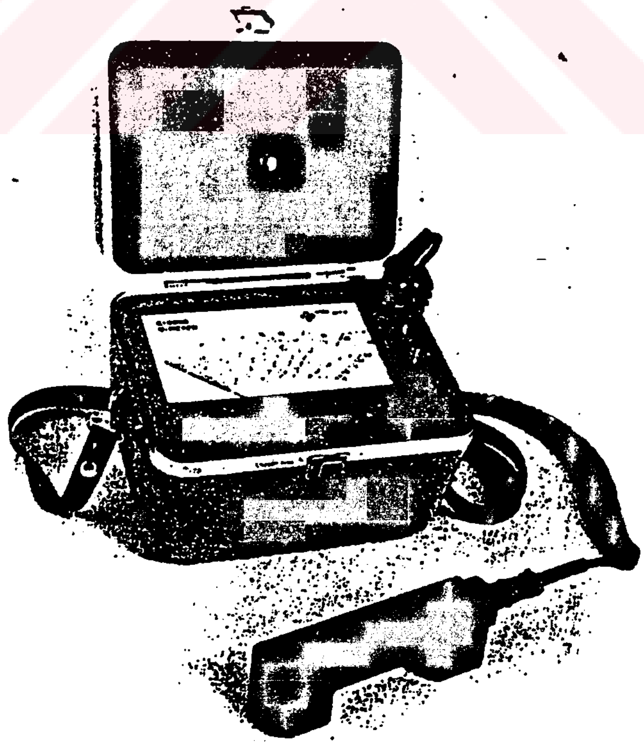


Fig. 2.4-Pachometer (reinforcing bar locator)

2- Soil investigations:

(i) inspection pits,

(ii) geotechnical borings, sampling, testing.

3- Concrete or mortar strength evaluation:

(i) core's taking and testing,

(ii) combined non-destructive methods.

4- Reinforcement characteristics:

(i) ultrasonic and gamma radiography methods (defects, correctness of detailing) as well as magnetic methods,

(ii) strength and ductility characteristics of steel samples (in-situ and/or laboratory estimations),

(iii) electrical potential measurements (state of chemical attack).

Also, special tests and measurements may be carried out in order to complete the information regarding the effects of aging-environmental actions or fire, such as:

- chemical analysis of concrete and/or steel,
- detection and determination of chloride content in hardened concrete,
- spraying with solutions of suitable color indicators (acid-base) for determining the depth of carbonation (color changes),
- special thermoluminescence examinations of aggregates for determining the maximum temperatures reached during afire, etc.

2.2 Criteria for Evaluation of Overall Loading Tests:

The method for checking the structural adequacy of doubtful members is a load test. Generally, the overall loading tests for damaged and/or weak buildings are not able to yield information valid for large plasticdisplacements.[2]

This is due to the fact that all induced action-effects (accelerations, velocities and displacements for dynamic tests or levels of loadings and displacements for static tests) are limited.

2.2.1 Vibrational Characteristics:

The main types of dynamic tests suitable for existing structures may be summarized as follows:[4]

a) Free-vibration tests:
   1- initial displacement tests,
   2- initial velocity tests.

b) Forced-vibration tests:
   1- steady-state sinusoidal excitations,
   2- variable frequency sinusoidal excitations,
   3- transient excitations.

Although the free-vibration tests may appear to have the virtue of a relative simplicity, forced-vibration tests will usually produce more complete and more accurate information; the increased complexity and cost of such forced-vibration tests are usually justified. Such test results can be used to estimate the natural frequencies and damping ratios. It is known that the estimated dynamic characteristics vary with the type of test, as well as with the amplitude of excitation causing vibration. In the following sections the above mentioned tests will be briefly outlined.

2.2.1.1 Free-Vibration Tests (see R. Wiegel. 1970)[4]

1- Pull-back and quick-release tests:

The simplest type of dynamic test consists of deforming a structure by pulling-on it with a cable (or a system of cables) which is then suddenly released (by means of electromagnetic devices), thus causing the structure to perform free vibrations about its static equilibrium position. By recording the vibration curve vs time, the natural period of the structure can be directly determined. Because of energy dissipation in the structure, the vibration amplitudes will decay, and from the ratios of amplitudes of successive cycles of motion the damping can be calculated.

A difficulty in such tests is to apply the pull and release it in such a way that the structure will vibrate in one plane only. What often happens is that two different modes of vibration may be simultaneously excited. If the periods of the two modes are close together, as it will be the case with the structures which usually have at least an approximate structural symmetry, the resulting motion will show beat phenomena, making it difficult to obtain any meaningful measure of damping.

Finally, another advantage of these tests is the following up of the behavior of the structure during several levels of horizontal load.

2- <u>Impacts, rockets, etc</u>:

Free vibrations can be also set up in a structure by initial velocities rather than by the initial displacements described above in the pull-back tests. This can be done by impact forces, caused by falling weights or by a pendulum that can strike a horizontal blow:

The swinging mass is hanged from a cantilever on the roof of the building or the boom of a crane, it is deviated a certain angle and then it impacts the building at the level of one of its rigid diaphragms.

Impulsive loads for structural tests also have been generated by explosive cartridges or by small rockets. In such impulsive tests the total time duration of the applied force is preferably short compared with the natural periods of significant structural modes, so that the resulting motions are functions of the total impulse or the initial velocity rather than of the force magnitudes. Also, the main disadvantage of these is the possibility of local damages.

2.2.1.2 **Force-Vibration Tests** (see R. Wiegel, 1970)

1- Steady-state sinusoidal excitations:

(i) Vibrators:

A steady-state resonance test involves the application to the structure of a sinusoidally varying uni-directional force (by means of rotating eccentric weight vibration generators, electromagnetic vibration exciters or hydraulic-tybe vibration actuators) whose period can be held accurately constant at one value, while measurements are made of the resulting motion of the structure. The period is then adjusted to a new value, the measurements are repeated, and so on to describe the whole period-response curve. Thus, by measuring the amplitude of motion of the structure at various periods covering the whole range of natural periods of the structure, the resonance curves can be plotted.

From these resonance curves, accurate values of natural period and damping can be obtained. By varying the magnitude of the exciting force as well as its period, various non-linear characteristics of the structure can be studied.

(ii) Man-excited vibrations:

All of the above methods of resonant vibration testing require relatively complex equipment and hence are limited in various ways to very special field conditions. There is thus a need for simplest tests that would give some information on building characteristics without the elaborate preparations that often make more complete dynamic tests impractical.

The method of man-excited vibrations (with its very low vibration amplitudes) is most useful for multistory buildings with relatively low natural frequencies

2- Variable frequency sinusoidal excitations:

In steady-state resonance tests of the type described above, the object is to hold the exciting frequency fixed for time sufficient so that all transient motions have died out and a uniform steady-state motion has been established. Because of the practical difficulties of holding fixed frequencies, there has been considerable study of the use of continuously varying frequency excitations.

The advantages of such tests are, first, the absence of a need for an elaborate speed control and, second, the rapidity with which the whole test can be carried out (and so, these "run-down" tests are very useful in preliminary investigations). The disadvantage is the difficulty of analyzing the data to find energy dissipation in the system.

3- Transient excitations:

(i) Microtremors:

Whereas large earthquakes are infrequent, there are many small earthquakes and there is a continuous motion of the ground caused by microseismic activity and by various man-made disturbances such as machinery, vehicle traffic, etc. Although these motions are usually of a very low magnitude, they can be used in certain circumstances to throw some light on structural properties

Because of the very low levels of vibration involved, methods of data reduction based on autocorrelation techniques have been developed to separate information from background noise.

Due to the fact that the microtremors cover a large range of frequencies, the estimation of natural frequencies of building (rigid, flexible or with moderate stiffness) is very good; on the other hand the estimation of damping characteristics is not so reliable.

Finally, it has to be noted that in case of R.C. buildings with well constructed infill walls (exterior and interior) the results of this method could be misleading, due to the high values of initial stiffness.

ii)- Wind-excited vibrations:

A useful source of excitation for natural period determination of full-scale structures is the wind. Even on a relatively quiet day, there are usually enough gusts of wind to excite a tall building into measurable vibrations at its fundamental period. Because of the very low level of excitation and the complex nature of the exciting force, the same type of data analysis based on autocorrelation techniques that was mentioned above for microtremor vibrations may also be applied to wind-excited motions.

Many structures for which wind-excited vibration tests are feasible, also would seem to be suitable for the type of man-excited forced vibration tests discussed above. In fact, wind excitation may interfere with such man-excited tests or with free-vibration decay tests. In such cases the operators will need to wait for a sufficiently quiet air condition to obtain undisturbed data.

iii)- Explosions:

The characteristics of ground motions during an explosion are very similar to those during an earthquake, although the accelerograms and acceleration response spectra are poor in long periods and so the results for flexible buildings are not so reliable.

2.2.2 Vertical Load Tests:

(proof-load tests)

In such tests, vertical loads higher than normal vertical service loads are applied uniformly and slowly on a R.C. structure, and the deformation characteristics of flexural elements are measured. The procedures of these loading tests and the acceptance criteria are specified by national standards.

The most commonly used criterion for acceptance is the ratio of residual to total deflections ($\lambda = a_{res} : a_{tot}$). This criterion is a typical case of a rule of thumb, intended to check the safety of the R.C. elements (slab and beams) tested. But, in fact, there is not a clear correlation between deformability and strength.[2]

Nevertheless, it is undeniable that λ -ratios tend to increase with the level of loading.

In what follows, some comments on the λ -ratio criterion are discussed, which may serve for the refinements they are certainly needed. Subsequently, some rules related to another criterion, based on rates of increase of residual deflections, are presented.

In fact, combined criteria prove somehow more promising than the old λ -ratio criterion of acceptance. Nevertheless, further research is urgently needed, both theoretical and experimental, in order to improve the reliability of estimations made out of load tests.

<u>The λ -criterion:</u>

1- It is clear that for under-reinforced (and quite so for balanced) sections, residual deflections are not influenced by concrete strength. Therefore, the λ -criterion can not be the general solution in case of doubts on concrete strength.
2- For the above mentioned sections (which constitute the majority of cases), the λ -ratio depends strongly on the initial level of loading- the dead load level. **Fig. 2.5.c** gives a clear, qualitative though, indication of the sensitivity of the phenomenon. The higher the initial level, the larger the λ-values are expected to be.
3- A rapid increase of λ -ratio is expected just after the middle level between M_{cr} and M_{st} (first cracking moment and stabilized cracking moment,

respectively). Afterwards, somehow constant values are expected. A similar trend is shown in **Fig. 2.6.** If this is so, the reliability of the criterion will diminish precisely near the level of test-loads where it is needed.

4- On the basis of the above mentioned fact, higher M_{cr}-values of prestressed concrete elements may produce much lower λ-values than the expected ones. From this point of view, the limit values for prestressed concrete elements provided by some codes are rather high.

5- Notwithstanding all these reserves, the λ-ratio is still very roughly increasing in function of load level, but it is very scattered. This is a fact which further diminishes the already poor reliability of the criterion.

<u>The rate of increase of residual deformations:</u>

1- Empirical diagrams of the unloading stiffness K' (slope of the secant of unloading curves) as a function of loading level, may in principle serve as a basis for the estimation of ultimate moment. Such an example is shown in **Fig. 2.7** and it is similar to the expected K' - diagram of Fig. 2.10.c. The scattering is, however, so large that any practical application is impossible, although this scattering is comparable to that of the λ-ratio.

2- Subsequently, instead of absolute values it is reasonable to examine the rate of change of quantities related to the loading level. Residual deflections seem to be the most expressive quantity in this connection. An indication of the possibilities offered for the determination of

M_{st} and M_o (stabilized cracking moment and moment corresponding to the elastic limit of steel, respectively) is shown in **Fig. 2.5.**

In a similar way of thinking, the following ratio could possibly be promising, as explained in **Fig.2.8:**

$$\xi = 1 - (a_{res} : a_{res,I})$$

Experimental data show that ξ-ratio has a maximum near M_u, consequently its practical usefulness is limited to cases of heavily damaged structures. A considerable improvement of the significance of information taken out of load tests would be achieved if during these tests strains were also measured: Differential strain diagrams due to the additional loading may offer additional information about the stage of the structure (degree of plastic behavior, etc.).

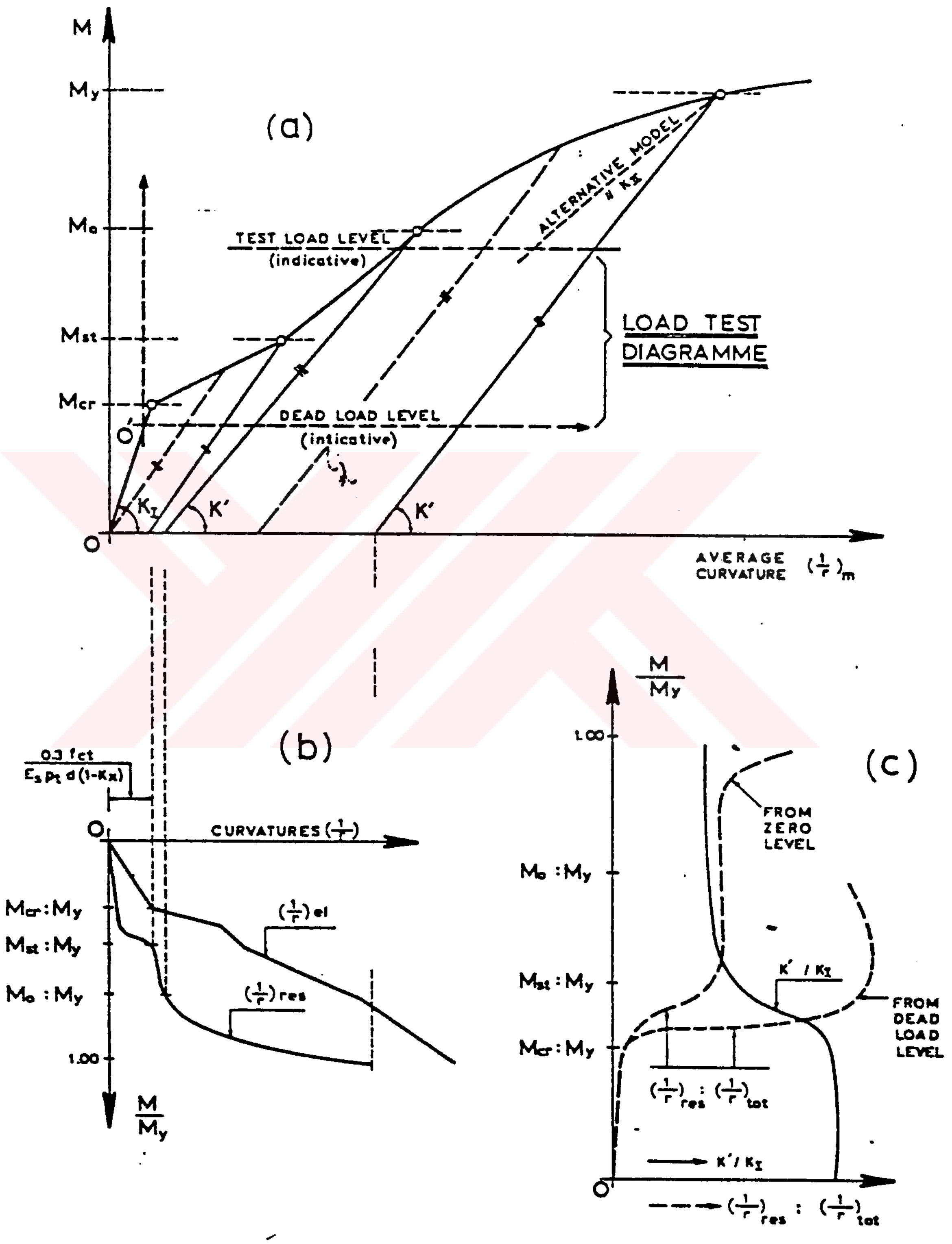


Fig. 2.5 Qualitative example for (unloading stiffnesses and residual curvatures

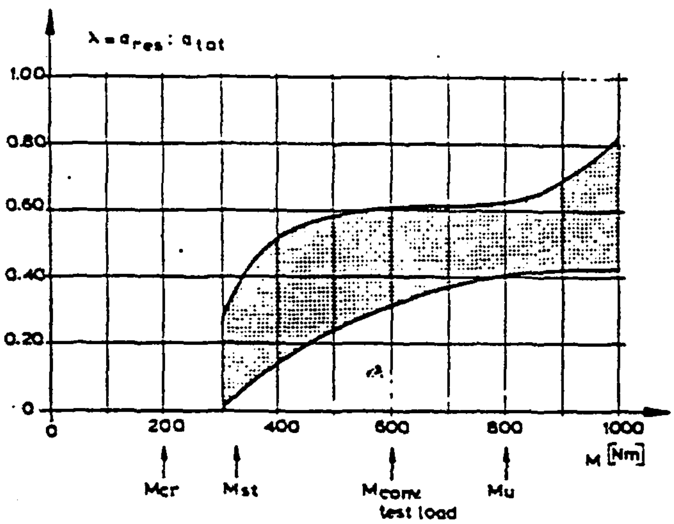


Fig. 2.6-Loading tests on R.C. model slabs

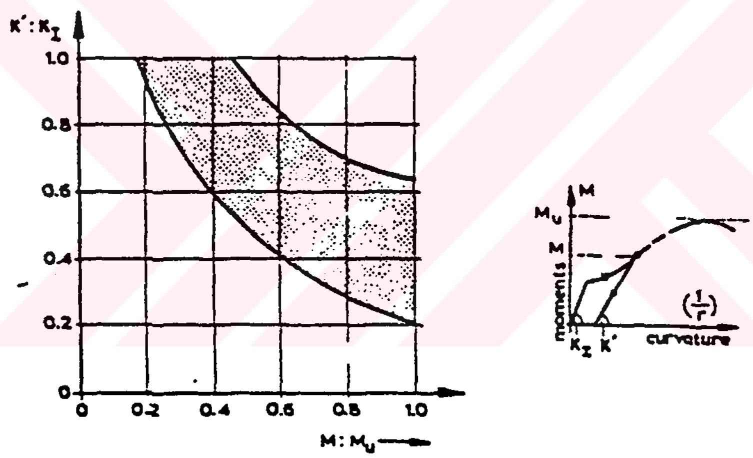


Fig. 2.7-Unloading stiffness of R.C. model slabs

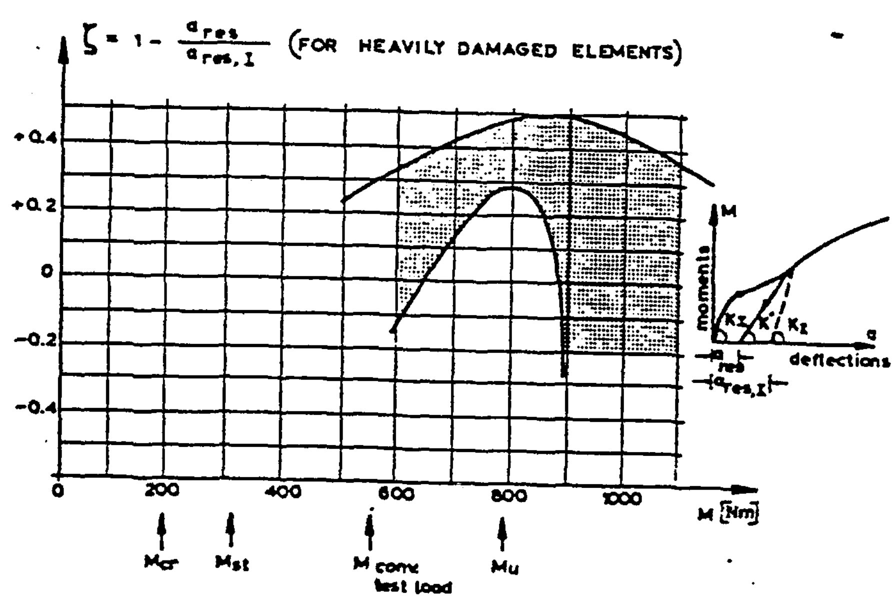


Fig. 2.8 Loading tests on R.C. model slabs

2.3 Estimation of The Intensity of Accidental Actions when Unknown (Trial Calculations to Reproduce Analytically the Damage-Morphology)

2.3.1 Principle of The Method:

Trial calculations may be carried out on the level of the structure, based on possible causes of the damage. The corresponding cracking patterns and widths, as well as residual deformations of vertical building-elements, are sought; that load intensity is retained which yields an overall pathological image as close as possible to the actual one.[2]

In the special case of a structure damaged by an earthquake, several seismic load intensities are tried, starting from an estimated value of the base shear-coefficient.

A much simpler approach confines itself to the level of damaged building-elements: Appropriate action-effects are sought, able to reproduce the actual damage or residual deformations observed on each particular building-element.[2] The following data may be helpful:

1- Cracking patterns (nature: flexural or shear, extent: length of area affected, etc.) and widths, for both structural and secondary elements.
2- Signs of local compressive failure (concrete popping, reinforcement buckling).
3- Residual deflections of vertical elements.

2.3.2 Illustrative Examples:

In the next paragraphs tow rather over-simplified examples are presented. They are only intended to illustrate the steps of an application of the above method in tow extremely simple cases; hence, they cannot be followed up in every real situation under complicated conditions.[2]

2.3.2.1 Damages Due to Earthquakes:

The 4-storey building shown in **Fig. 2.9** was damage by a severe earthquake. All the corner columns Of the ground floor have been cracked. The pattern of the cracks shows that the columns suffered of excessive moment intensity at their top and bottom. At first place, a structural analysis of the structure is carried out, in order to find out the normal forces which act on the damage d columns. Then, the moment-curvature diagram of the sections of the damaged columns **Fig. 2.10** is found; similarly the maximum moment the columns were able to withstand before the earthquake is also determined.

Afterwards, trial calculation start: since the illustrated crack pattern is developed, the moment due to the earthquake should be between the values M_B and M_U.

Based on seismological data and on our experience let's assume that the base shear-coefficint was equal to $c \approx 0.12$. Then:

1- $N_{tot} \approx 9300.0$ KN, $V_{tot} \approx 9300.0 \times 0.12 \approx 1116.0$ KN

The distribution of the horizontal force is carried

out according to the flexibility of each column.
Thus, shear force and moment suffered by column A:
2- $V_A \approx 60.5$ KN, $M_A \approx V_A(h/2) \approx 145.2$ KNm.
This figure is very close to M_u (≈ 147.5 KNm),
and therefore the damage at the corner columns A
may be explained.
The assumption of $C \approx 0.12$ was right.
If not, we should start the whole process again
assuming another value for C.

2.3.2.2 Damage Due to Differential Settlements:

The portal frame shown in **Fig. 2.11** developed the crack pattern shown in **Fig. 2.12**. The nature of the cracks indicates that the column AC suffered of excessive moment developed at its top due, most probably, to the moment developed from differential settlement of the foundation **Fig. 2.13** and added to the moment due to the static loads. After further inspection and carrying out appropriate measurements, a differential settlement of the base of the right-hand column was found equal to 80 to 90 mm **Fig. 2.13.**

A structural analysis is carried out first, in order to find out the *normal forces* which act on the damaged column and the magnitude of the moment due to static loads at its top:

1- $N \approx 400.9$ KN, $M \approx 75.0$ KNm

Afterwards, the moment-curvature diagram of the section of the damaged column is computed **Fig. 2.14** the maximum moment the column was able to withstand before the damage is also evaluated. The developed crack pattern **Fig. 2.12** indicates that the sum of the moment due to the static loads and due to differential settlement should be between M_s and M_u **Fig. 2.14.**

The moment in the beam is between level I and S **Fig. 2.14;** thus its stiffness has to be taken as equal to K_{II}. Hence, for an average settlement $\delta \approx$ 85 mm, the moment at the top of the column is:

2- $M \approx 70.3$ KNm.

Thus, the total moment at the damaged section is equal to:

3- $M \approx 75.0 + 70.3 \approx 145.3$ KNm.

This figure is very close to M_u ($\approx$ 147.5 KNm), and therefore the **damages may be explained by such a settlement.**

The above calculations show that the estimation of $\delta \approx$ 85 mm was right. In an opposite case, we should start the whole process again, assuming another value for δ and checking it again.
(Note: The above portal frame was supposed to be adequately reinforced to resist shear).

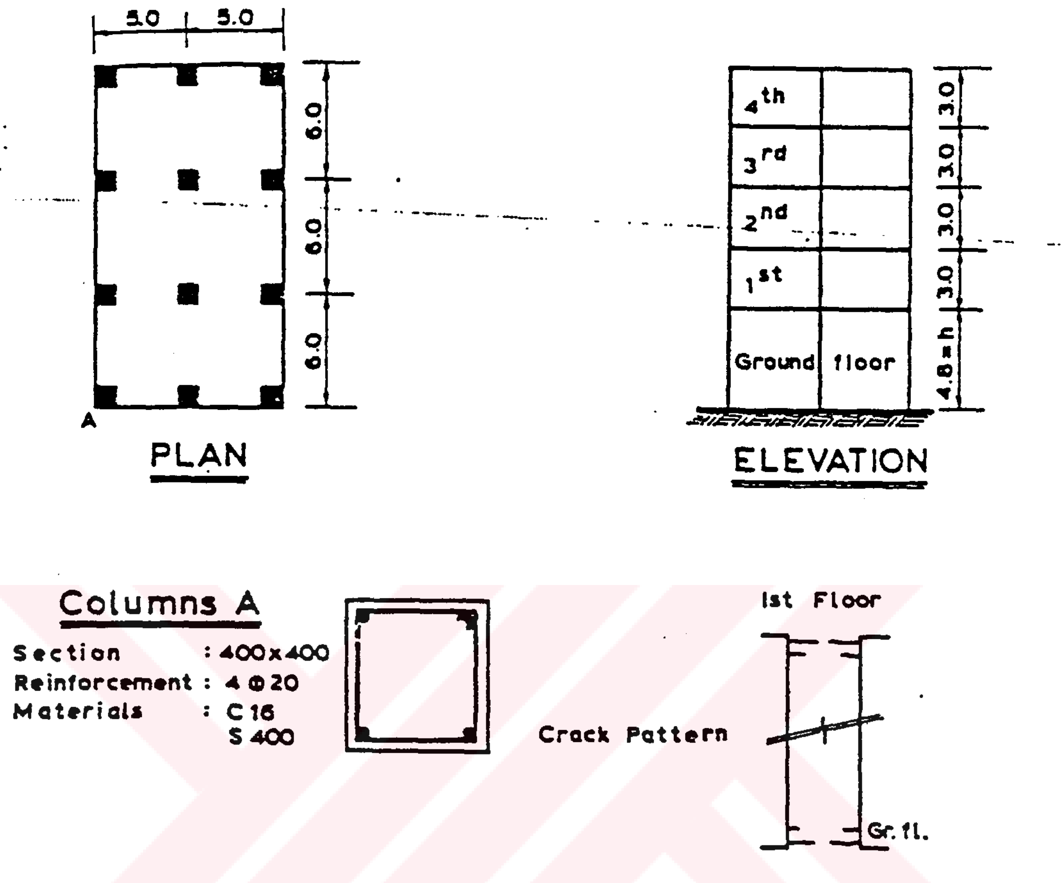


Fig. 2.9 4-story R.C. building damage by a severe earthquake

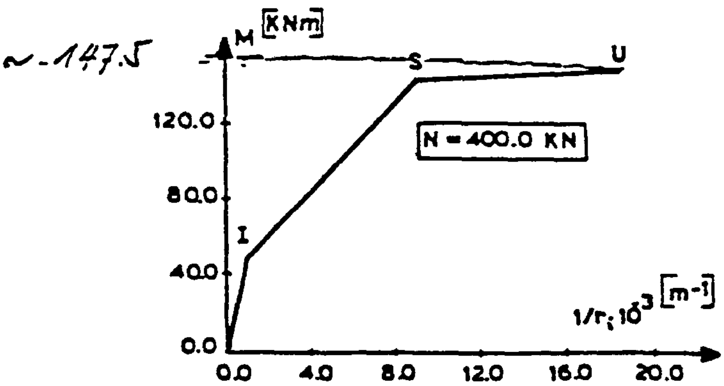
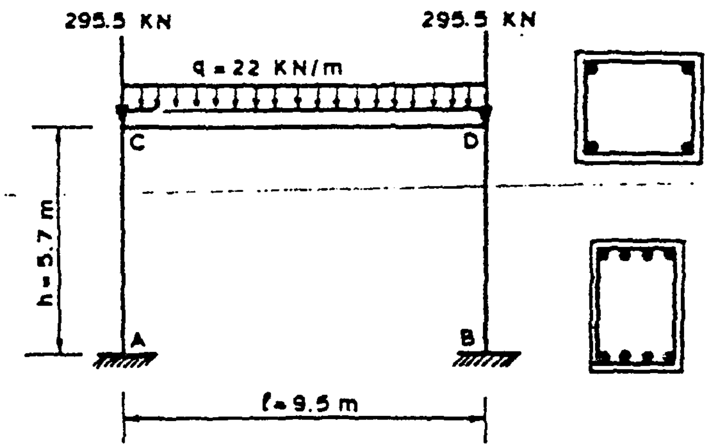


Fig. 2.10 Moment-curvature diagramme of the damaged columns



Column's section

Section: 400x400

Reinforcement: 4Ø20

Beam's section

Section: 300x700

Reinforcement: 4Ø16 = A_s

4Ø16 = A'_s

Materials: C16

S400

Fig. 2.11 R.C. portal frame.

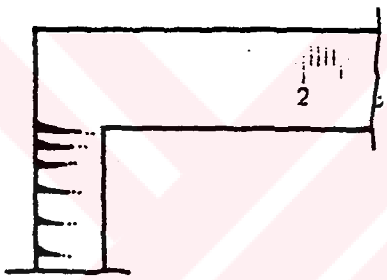


Fig. 2.12 Crack pattern at the top of left-hand column

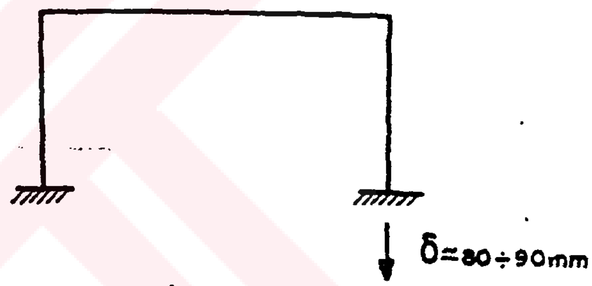


Fig. 2.13 Differential settlement of the base of righthand column

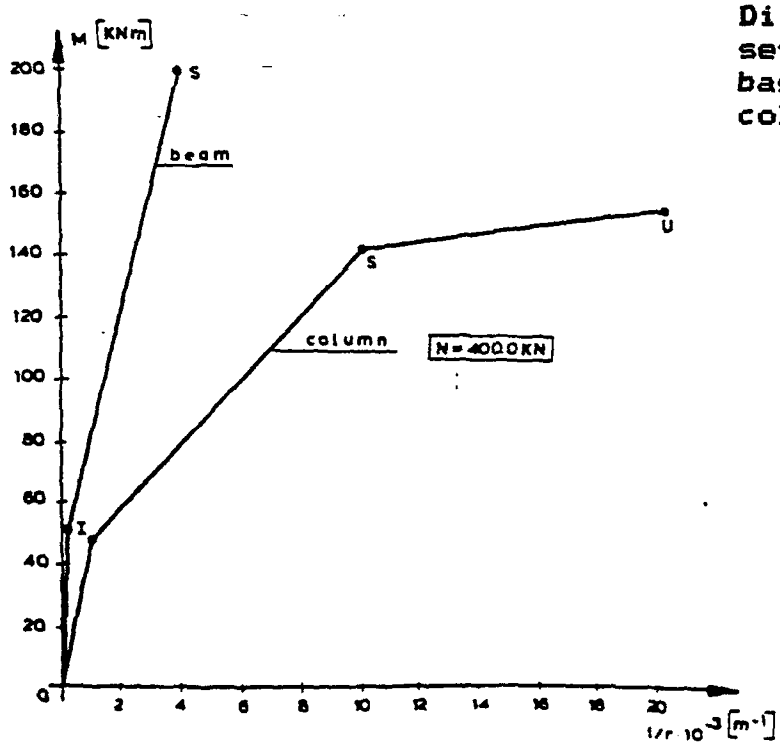


Fig. 2.14 Moment-curvature diagrammes

Chapter 3

EMERGENCY MEASURES FOR TEMPORARY PROTECTION

3.1 General

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

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

The temporary protection should be provided in the following:

1- Vertical support for failed or severely damaged columns or bearing wall.
2- Lateral counterforts or wall braces should be provided for walls which may fall laterally or when there is doubtful stability of the structure against horizontal forces.
3- Diagonal braces of structural frames can also be installed to provide bracing of structure as a whole.

The design of emergency measures for temporary protection must usually be done quickly without the normal time period available for design procedures. The designer should pay special attention to the following:[1]

1- The shear strength of the supported beams, indicated as section t-t in **Fig. 3.1**, must be carefully checked
2- Transmission of the temporarily supported loads to the adjacent elements as well as to the soil beneath the structure.
3- Temporary foundations may be required
4- The distance between the supports and the damaged member must be minimum.

Available materials must be utilized and only crude, approximate analysis are usually performed to determine the order of magnitude of loads and stresses. Due to the need for prompt action, judgment and ingenuity must replace detailed analysis.

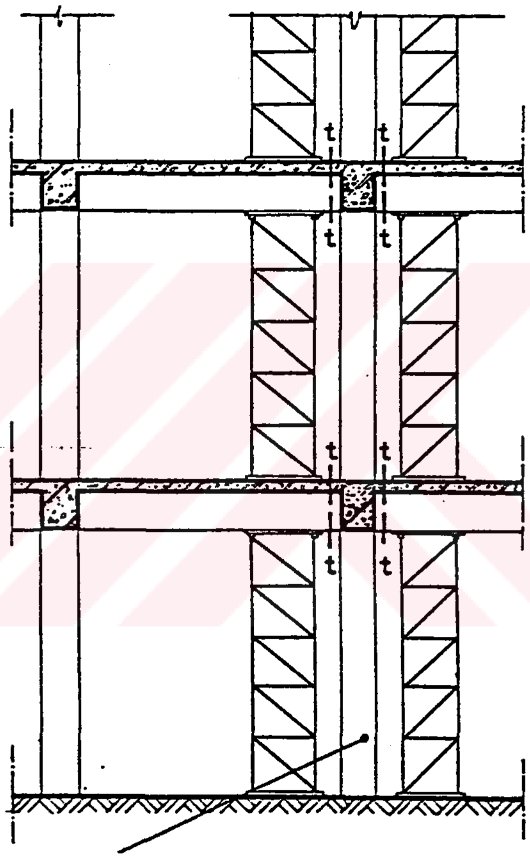


Fig. 3.1 Temporary vertical support

Depending on the means available, the seriousness of the damage and the size of the structure, a suitable method of supporting can be selected for vertical load support of damaged or failed members, techniques such as industrial scaffolding, tree logs, steel profiles, or grillage logging can be installed. For lateral support of wall, or the structure as a whole, lateral wall bracing or frame bracing can be installed The description of these methods is given in the following paragraphs in more details.

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

3.2 Method for Supporting Vertical Loads:

Industrial-Type Shoring and Scaffolding:
In the case of very light loads, independent supports utilizing standard industrial-type shoring can be used with bearing capacity up to 2 tons and for height of about 3.00 ton. Industrial -type scaffolding **Fig. 3.2** can be used in case of minor damage and for small loads, or in cases where flexural members (beams or mainly slabs).[1]

Timbers, Tree logs and Telephone poles:
Timber, such as tree logs, sawn timbers or telephone poles can also be used for supporting vertical loads, wood used must be free of defects

(knots, hollows, etc.). The efficiency of the timber or log section must be checked on the bases of an approximate of the load and of the maximum permissible stress of the wood along the loading direction. When the loads are sufficient to require two timbers or logs on each side of the vertical element, use at least two approximately 25 cm diameter logs or 20 cm sawn timbers, which should be interconnected with wood planks and/or stitching dogs, put in pairs, in the from of the letter X, as shown in **Fig. 3.3.**

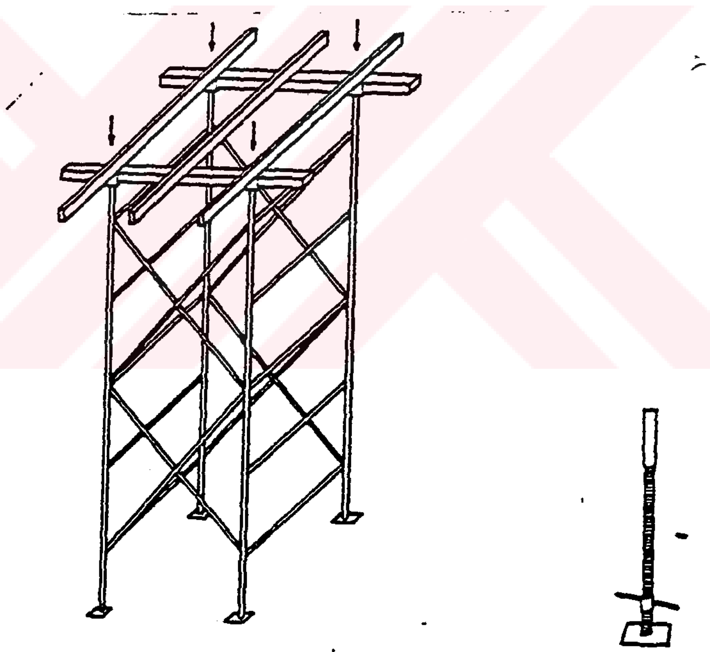


Fig. 3.2 Industrial-type scaffolding

__Steel Profiles__:
Steel profiles can be used for vertical support in the same manner as tree logs. Steel sections require bearing plates of sections both top and bottom and must be properly wedged. Steel

profiles assembled around the perimeter of a damaged column similar to permanent jacketing with steel profiles can be utilized for temporary support.

Timber Grillages:

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

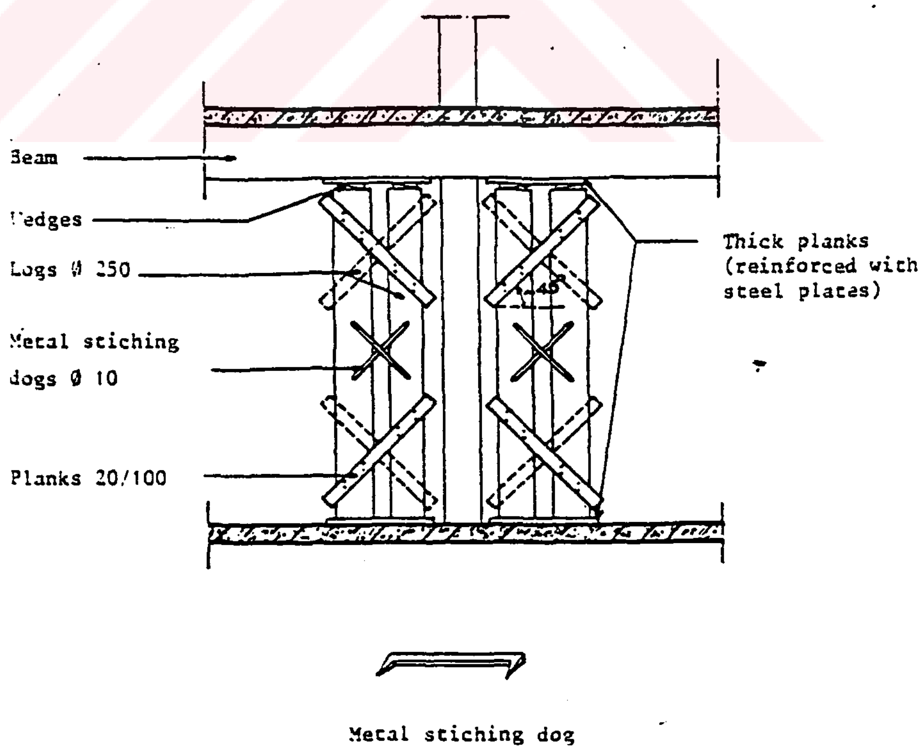


Fig. 3.3 Vertical support showing planks, logs and metal stitching dog

3.3 Methods for Providing Lateral Support:

Lateral Wall Bracing:

Lateral wall bracing can be installed to prevent the wall from falling outwards, as shown in **Fig. 3.4.** The members of the bracing can be timbers, logs or steel profile. The diagonal member should be placed as an appropriate angle for stability with a suitable foundation in the ground to make the brace effective. If it is impossible to install such a diagonal brace, an alternate form of providing equivalent wall bracing is installing internal tension ties within the structure with steel anchorages and bearing elements on the outside of the wall, as illustrated in **Fig. 3.5.**

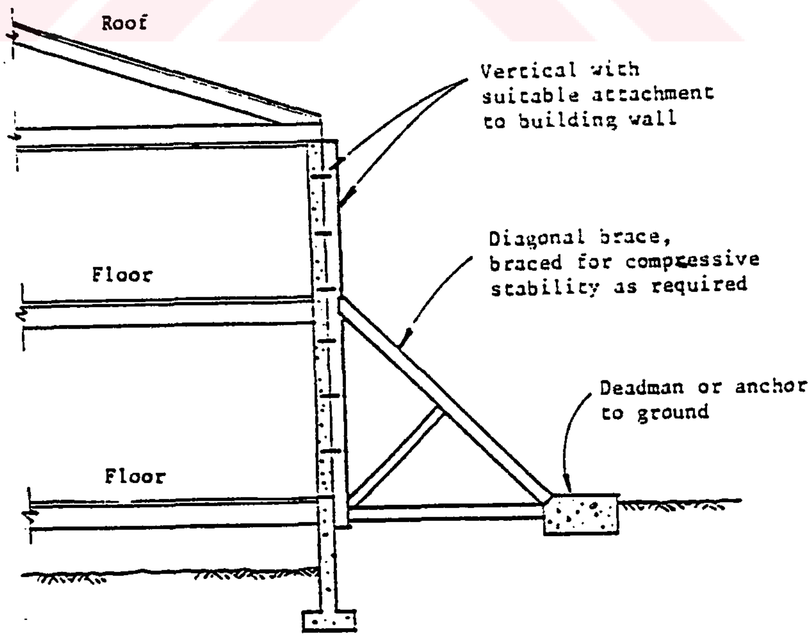
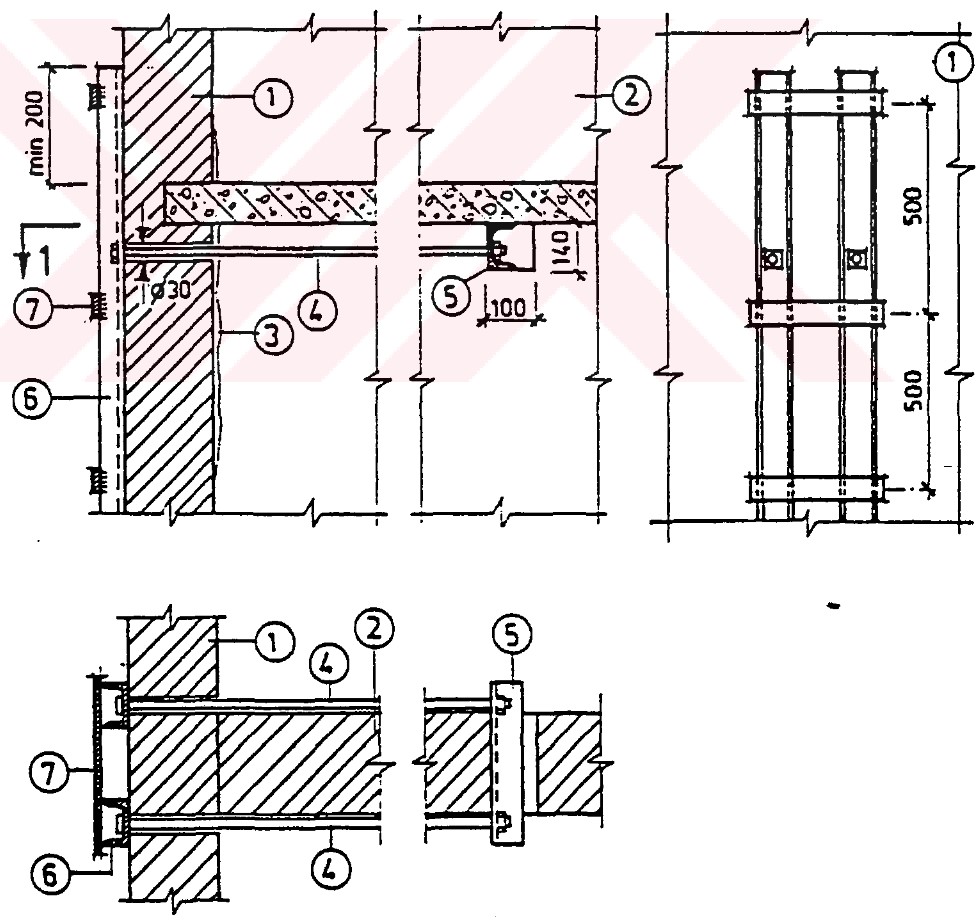


Fig. 3.4 External lateral wall bracing

<u>Frame Bracing</u>:

When frame buildings have been damaged to earthquake, it is necessary to install some form of temporary lateral bracing to prevent the building as a whole from collapsing during the continued seismic activity. Frame bracing can consist of timbers, tree logs or steel profiles of sufficient strength considering their potential for buckling. Bracing members are installed on a diagonal between columns on a frame line as shown in **Fig. 3.6.**



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

Fig. 3.5 Interal lateral wall bracing

3.4 Wedging Techniques:

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

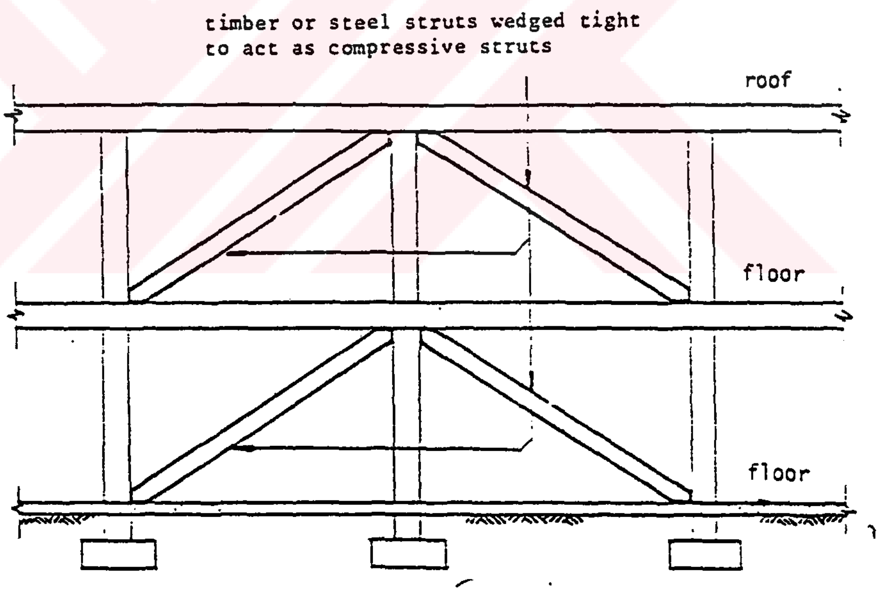


Fig. 3.6 Frame bracing

Chapter 4

ESTIMATION OF RESIDUAL STRUCTURAL CHARACTERISTICS

4.1 Analytical Methods:

4.1.1 Constitutive Law of Materials:

4.1.1.1 Due to Mechanical Actions:

a) Steel

In most cases of engineering interest, the way of formulating the steel behavior is crucial to the success of any analytical model for reinforced concrete structures. The post-cracking response may largely be governed by the reinforcement characteristics. Most of the analytical formulations being proposed so far are established by seeking close fits to experimentally obtained stress-strain curves rather than being based on any basic knowledge of the material on the microscopic level. It is believed that such basic knowledge is not available to a sufficient degree yet.

(i) Monotonic stress behavior:

Typical stess-strain curves for steel bars used in reinforced concrete construction are shown in **Fig. 4.1** for monotonic loading. It is generally assumed that the stress-strain curves for steel in tension and compression are identical. This has been reasonably verified by tests.

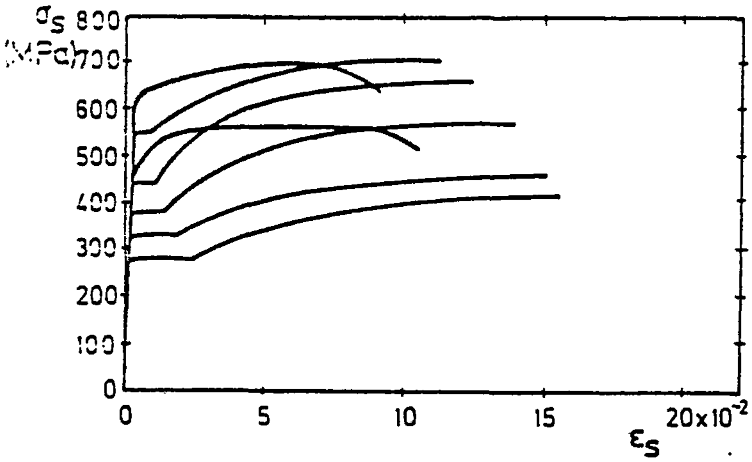


Fig. 4.1 stress-srain relationship of steel bar

(ii) Repeated stress behavior:

A typical stress-strain curve for steel under repeated loading is shown in **Fig. 4.2**. The monotonic stress-strain curve gives a good idealisation for the envelope curve for repeated of the same sign.

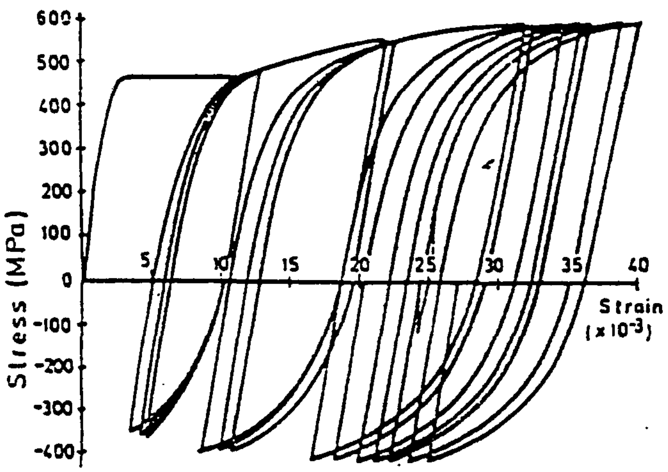


Fig. 4.2 Stress-strain curve for steel under repeated loading of the same sign

(iii) <u>Reversed cyclic stress behavior:</u>
A typical stress-strain curve for steel under reversed loading is shown in **Fig. 4.3**, exhibiting the Bauschinger effect, in which under reversed loading the stress-strain curve becomes nonlinear at a stress much lower than the initial yield strength. Reversed loading curves for steel are of fundamental importance when considering analytically the deformation characteristics of reinforced concrete members under seismic loading. Several analytical models have been proposed to reflect the above behavior:

1- Elasto-plastic model.
2- Multi-linear model.
3- Menegotto and Pinto model.
4- Kato, Akiyama and Yamanouchi model.
5- Kent and Park model.
6- Aktan, Karlson and Sozen model.
7- Ma, Bertero and Popov model.
8- The mechanical sublayer (or overlay) model.

For more details about the above mentioned models see the C.E.B Bull. 161_"Response of R.C. critical regions under large amplitude reversed actions".

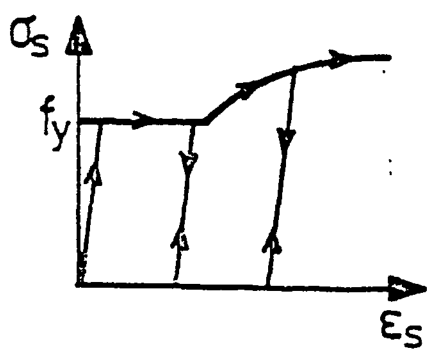


Fig. 4.3 Typical experimental stress-strain curve for steel under reversed cyclic loading

b) Concrete:

A large number of material models for approximation of the uniaxial, biaxial and multiaxial behavior of concrete under cyclic loading has been proposed during the two last decades.

(i) Uniaxial cyclic behavior of concrete:

Fig. 4.4 shows experimental results (Park R. and Paulay T. : Reinforced concrete structures) of stress-strain curves of concrete under cyclic loading. Many researchers have found that a unique envelope curve of the stress-strain curves for concrete can be defined which connects the starting points of unloading curves as well as the ends of reloading curves regardless of the load pattern, i.e. it is a limiting curve within which all stress-strain curves lie **Fig. 4.5.**

Many models have been proposed to simulate the behavior of concrete in uniaxial cyclic loading, some of them are suitable for research purposes, also some of them are suitable for design purposes.

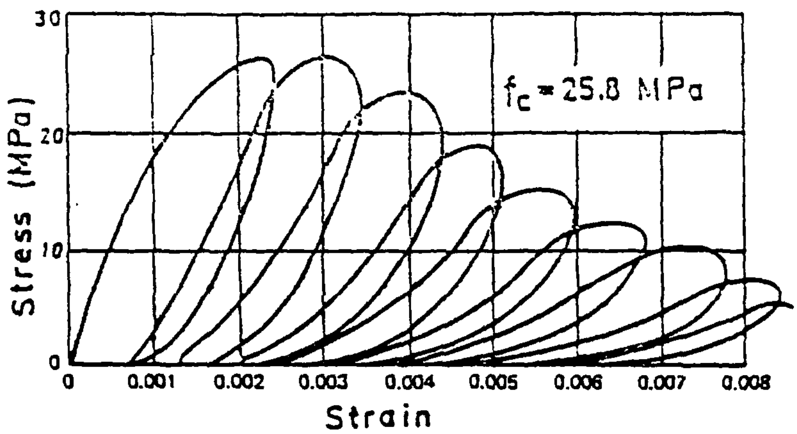


Fig. 4.4 Stress-strain curve for concrete cylinder under cyclic loading (Park. R and Paulay T. 1975)

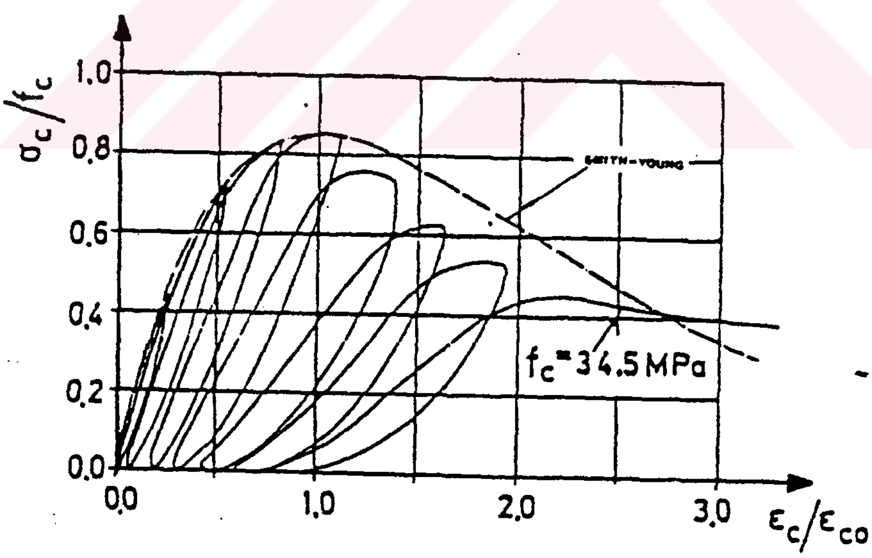


Fig. 4.5 Cyclic loading and envelope (Karsan and Jirsa 1969)

(ii) <u>Biaxial cyclic behavior of concrete:</u>

Darwin and Pecknold (1974) developed a model for the biaxial cyclic behavior of concrete, which is based on a nonlinear elastic formulation, i.e. by treating the material as an incrementally linear, elastic material. During each load increment the material is assumed to behave elastically, while between increments the material stiffness and stresses are corrected to reflect the latest changes in deflection and strain, using an adaptation of the initial stress method. The constitutive model describes a stress-induced orthotropic material behavior. The incremental constitutive relation between stresses and strain is obtained as:

$$\begin{bmatrix} d\sigma_1 \\ d\sigma_2 \\ d\tau_{12} \end{bmatrix} = \frac{1}{1-\nu^2} \begin{bmatrix} E_1 & \nu\sqrt{E_1E_2} & 0 \\ \nu\sqrt{E_1E_2} & E_2 & 0 \\ 0 & 0 & 1/4(E_1+E_2-2\nu E_1E_2) \end{bmatrix} \begin{bmatrix} d\varepsilon_1 \\ d\varepsilon_2 \\ d\gamma_{12} \end{bmatrix}$$

or, written in matrix notation

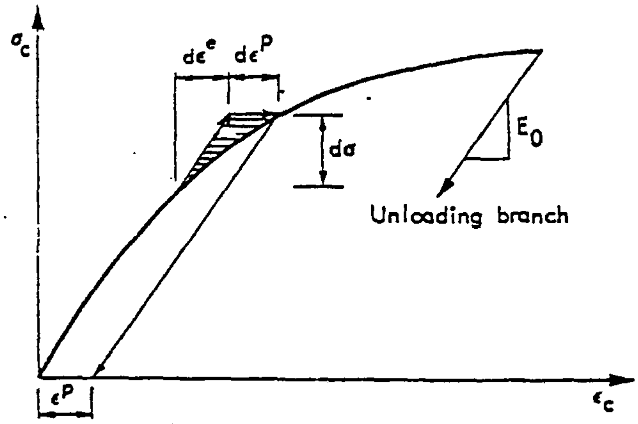
$d\sigma = D.d\varepsilon$

The term $D_{33}=1/4(E_1+ E_2-2\nu\sqrt{E_1E_2})$

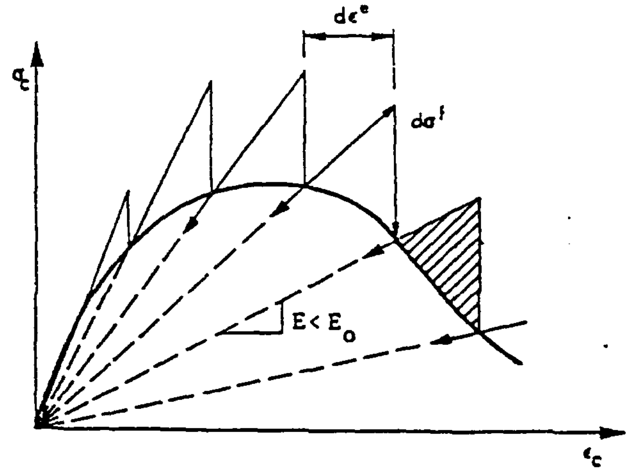
is chosen to make the constitutive relation direction-independent,and the constitutive matrix D is defines by only three quantities: E_1,E_2 and ν

(iii) <u>Bazant Z.P. and Kim S.S (1979)</u> developed a plastic-fracturing model for the multiaxial cyclic behavior of concrete. The basic idea in this model is that inelastic strains consist of one part due to plastic deformation and one part which is connected to microcracking in the concrete.

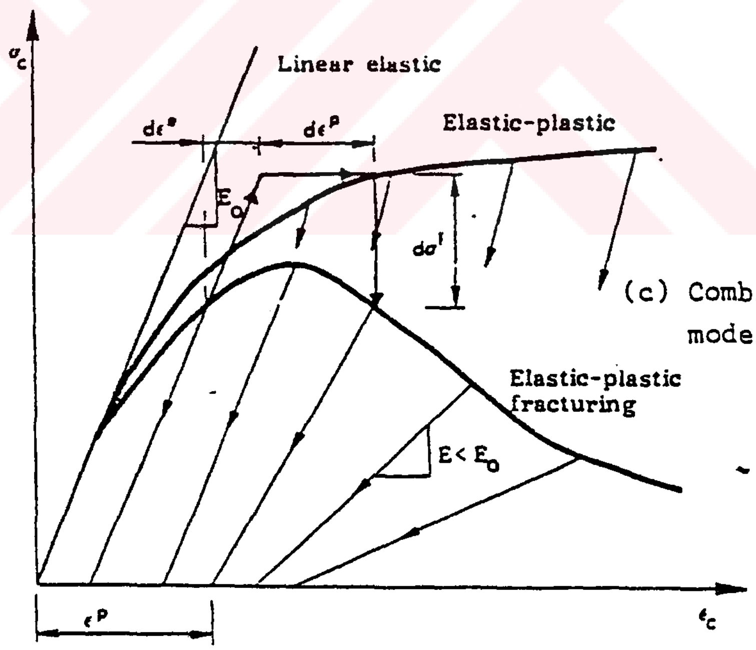
The characteristic features of the model can be illustrated as shown in Fig. 4.6. Fig. 4.6(a) represents an elasto-plastic model, where the strain increment consists of an elastic and a plastic part. Unloading in this model is linear and follows the initial modulus of elasticity. Since plastic slip can not explain a descending branch or strain softening of the stress-strain curve, such effects must be connected to micro-cracking or fracturing in the material. Dougill J.W. (1976) developed a model for a fracturing material where an elastic strain increment is followed by a fracturing stress decrement. This model is illustrated in F.g 4.6(b). The plastic-fracturing model combines these two models, and the result is shown in Fig. 4.6(c). This model is capable of describing both strain-softening and stiffness degradation during load cycling.



(a) Elasto-plastic model



(b) Fracturing model (Dougill 1976)(1.1-7)



(c) Combined Plastic fractu
model (Bazant, Kim 1979

Fig. 4.6 Basic concepts of plastic-fracturing model

c) <u>Bond:</u>

Local bond (τ) versus local slip (s) relationships along a bar inserted in a concrete elements, are of fundamental interest for the analytical investigation of reinforced concrete deformation, especially under cyclic loading.

(i) <u>Basic models of local bond-local slip behavior under monotonic actions</u>:

An attempt has been made for a synthesis of a physical model **Fig 4.7** regarding the characteristic values of the τ-S constitutive law under monotonic loading. This lead also to analytical expressions of the stresses τ_A, τ_B, τ_u, τ_r. Further details are given for the characteristic points of a monotonic τ- diagram **Fig.4.8.**

The shape of the local bond law is significantly influenced by the position of the bar during casting. The largest bond stiffness is reached for bars cast vertically and loaded against the setting direction of fresh concrete. Bars cast horizontally show a much smaller stiffness and a lower strength.

The local bond stress-slip relationship may vary along the embedment length. Bond stiffness and bond strength are 2-3 times larger in the interior of a specimen with tension forces acting on both ends of the embedded bar than towards the ends.

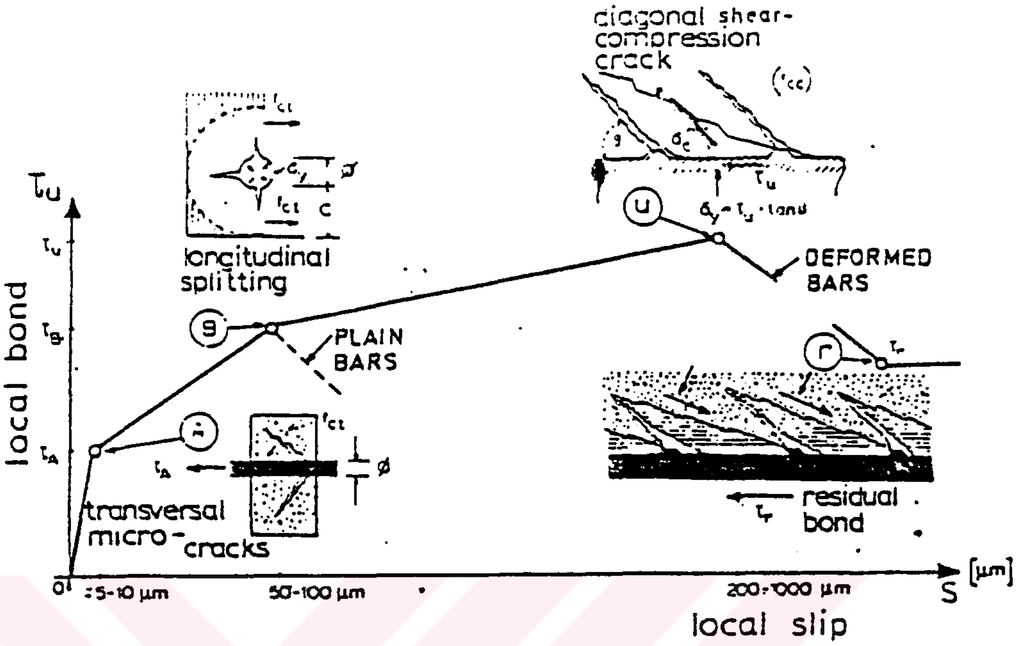


Fig. 4.7 Characteristic τ- values (Tassios T.P)

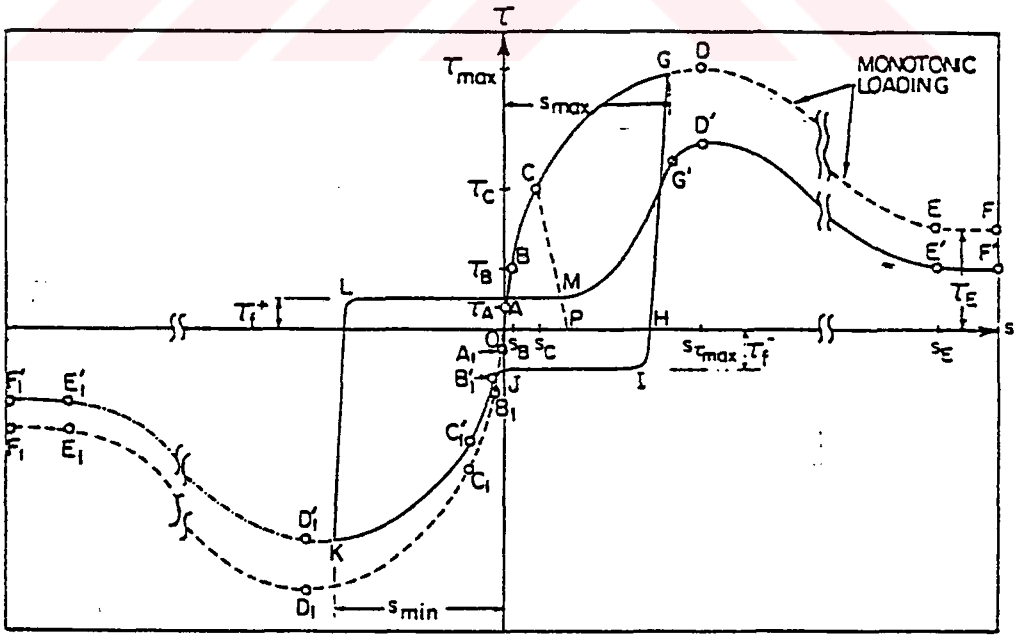


Fig. 4.8 Typical relationship between bond stress τ and slip S for monotonic and cyclic loading (Eligehausen R., Bertero V., Popov E. 1983)

(ii) Repeated loading:

Repeated load has a similar influence on the slip and the bond strength of deformed bars as on the deformation and failure behavior of unreinforced concrete load in compression. The bond strength decreases with increasing number of cycles between constant bond stresses. The slip under constant peak load and the residual slip increase considerably as the number of cycles increase **Fig. 4.9.** If no fatigue failure of bond occurs during cycling and the load is increased afterwards, the monotonic envelope is reached again and followed thereafter. That means, provided the peak load is smaller than the load corresponding to the fatigue strength of bond, a preapplied repeated load influences the behavior of bond under service load but does not adversely affect the bond behavior near failure compared to a monotonic loading.

(iii) Cyclic reversed actions:

The first analytical model of the local stress-slip relationship for cyclic loadings by Morita and Kaku as shown in **Fig 4.10.** The monotonic envelope, which are different for loading in tension or compression and for confined or unconfined concrete, are given by two successive straight lines which follow closely the experimentally measured curve. **Fig. 4.11** shows the bond model proposed by Tassios T.P. The monotonic envelope consists of six successive straight lines. More detail about this model explained in Bull. No. 161 (C.E.B) "Response of R.C. critical regions under large amplitude actions"

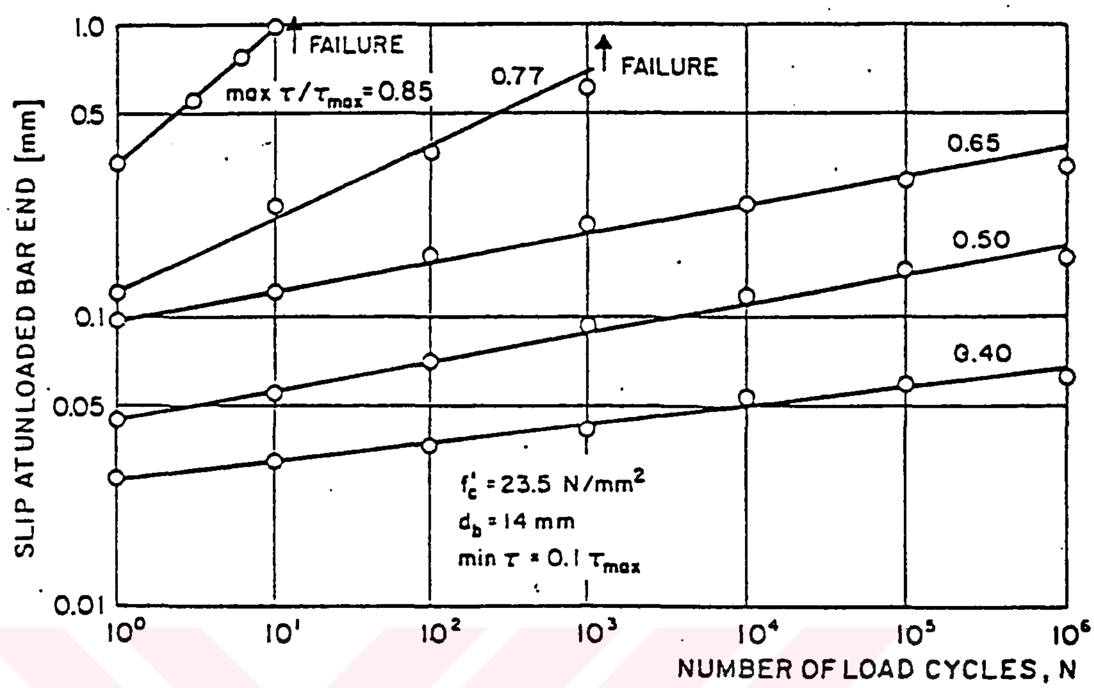
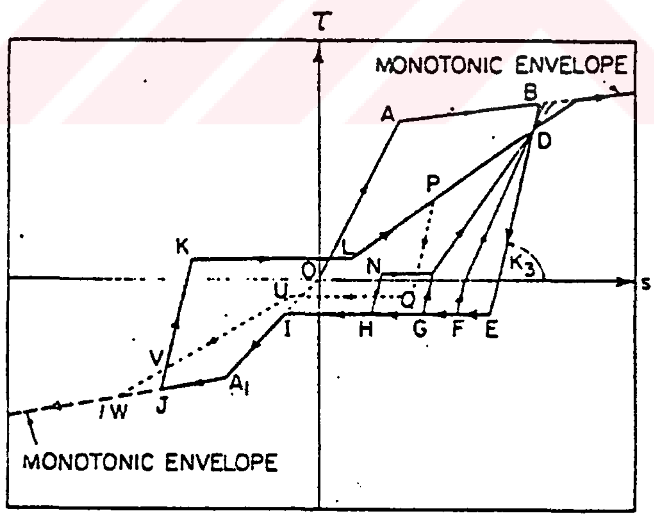


Fig. 4.9 Increase of slip at the unloaded bar end during cyclic loading as a function of the number of load repetitions (Rehm and Eligehausen 1977)



$\tau_D = \beta \cdot \tau_B$ $\quad$ $\tau_K = \alpha \cdot \tau_J$ $\quad$ $s_L = (s_B + s_J)/2$

$\tau_V = \beta \cdot \tau_J$ $\quad$ $\tau_N = \alpha \cdot \tau_H$ $\quad$ $s_U = (s_J + s_P)/2$

$\tau_E = \alpha \cdot \tau_B$ $\quad$ $\tau_Q = \alpha \cdot \tau_P$ $\quad$ $s_G = s_B/2$

$K_3 = 400\ N/mm^3$

$\alpha = 0.18$

$\beta = 0.9$ $\quad$ $s \le 0.05$ mm

$\beta = 0.9 - 0.44\,(s - 0.05)$ $\quad$ $0.05 \le s \le 0.5$ mm

Fig. 4.10 Analytical model for local bond sress-slip relationship proposed by Morita/Kaku 1973

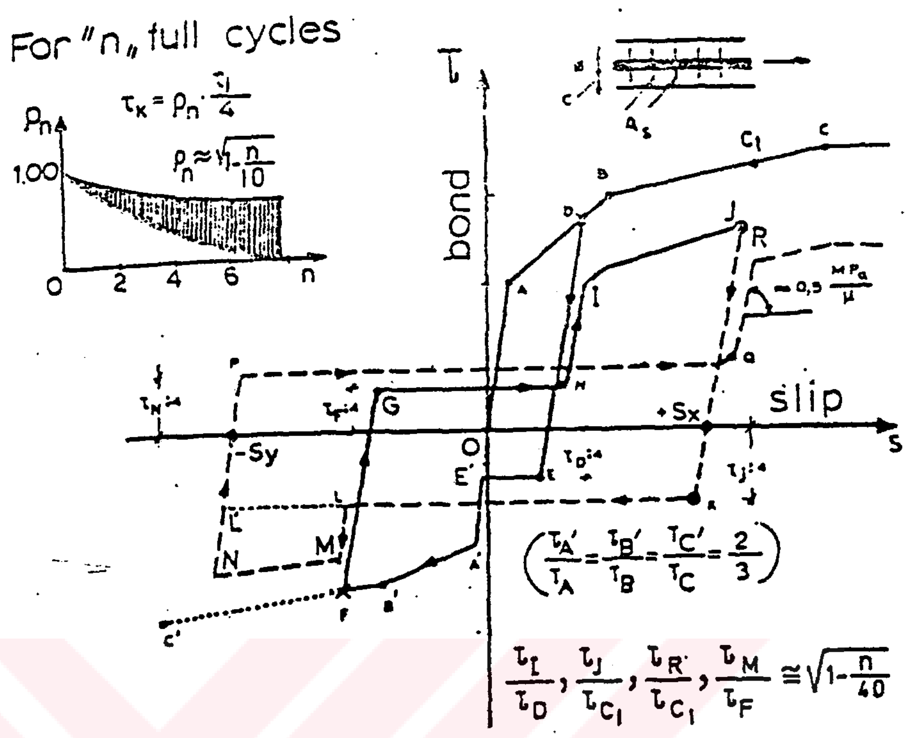


Fig. 4.11 A simplified model for reversed bond-slip constitutive law (Tassios T.P 1983)

(iv) <u>Factors affecting bond strength under cyclic loads:</u>

The main factors affecting bond behavior under cyclic loads are:

1- Concrete compressive strength;
2- Cover and bar spacing;
3- Bar size;
4- Anchorage length;
5- Rib geometry;
6- Steel yield strength;
7- Amount and position of transverse steel;
8- Casting position, vibration, and revibration;
9- Strain (or stress) range;
10- Type and rate of loading;
11- Temperature;
12- Surface condition (coatings).

4.1.1.2 After Fire:

The primary factors affecting fire development and temperature history in a compartment during a fire are the fire load per unit area, $Q = W\ M\ /\ A_t$; the ventilation parameter, $A_o\sqrt{h}$, the opening factor, $F = A_o\sqrt{h}\ /\ A_t$, which controls the rate of combustion; and the thermal properties of the compartment lining materials (Magnusson and Thelandersson 1974).[5,6,7] where:

A_o= total area (square meters) of door or window opening in the compartment;

A_t= total area (square meters) of the compartment bounding surface;

M = total mass (N) of combustibles in the room;

W = effective heat of their combustion.

The standard ASTM E119 fire test impose three conditions for failure for reinforced concrete components as follows:

1- Collapse of the component or failure to inhibit passage of flame or hot gases
2- Attainment of the limiting average temperature of 1100°F (593°C) in reinforcement or 800°F (427°C) in prestressing steel.
3- Rise of 250°F (139°C) in the average temperature of the unexposed surface of the test component.

The effect of fire on the components of reinforced concrete structures as steel and concrete as follows:

a) Steel:

The effect upon steel of high temperature and subsequent cooling is shown in **Fig. 4.12,** significant loss of strength may occur while the steel is at high temperature such that only 50% of the original yield strength remains at 600°C. Recovery of yield strength after cooling is generally complete, for temperatures up to about 400°C for cold-worked, and up to about 800°

C for hot-rolled steels.[2]

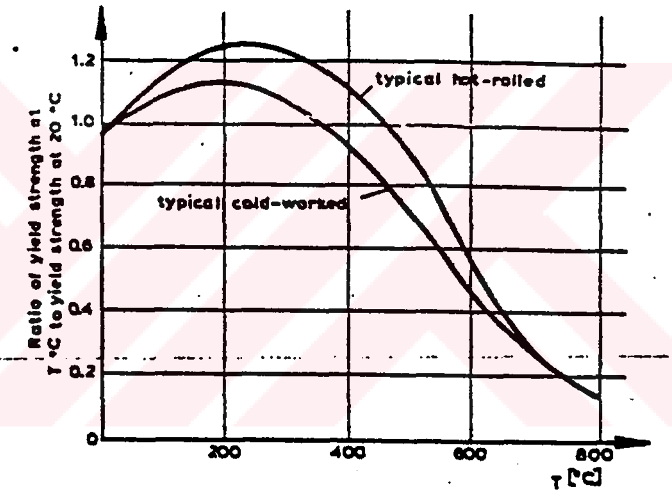


Fig. 4.12 Yield strength of steels tested at elvated temperatures [concrete society technical report No.15, London 1978]

b) Concrete:

Temperature changes have a marked effect on the thermal properties of concrete and a wide range of values have been reported by different investigators. This range due to the composite nature of concrete and varies with the properties of the aggregate and cement, their proportions, moisture content and also the history and age of the concrete. As reported by (Concrete Society , London, 1978, Technical report No. 15 "Assessment of fire damage concrete structures and repair by gunite") , the residual strength for dense concrete after cooling from various temperatures are shown in **Fig. 4.13.** For temperatures up to 300°C, the residual strength of structural-quality concrete is not severely reduced. On the other hand, temperatures greater than 500°C can reduce the compressive strength of structural-quality concrete to only a small fraction of its original value and such concrete is unlikely to possess any useful strength.[2]

Note: The effect of fire on local bond-local slip and the behavior of concrete after cooling have been reported by different investigators using a pull-out specimen. The measured bond strength follows a trend similar to that for the changes in the compressive strength of concrete up to 300°C-400°C

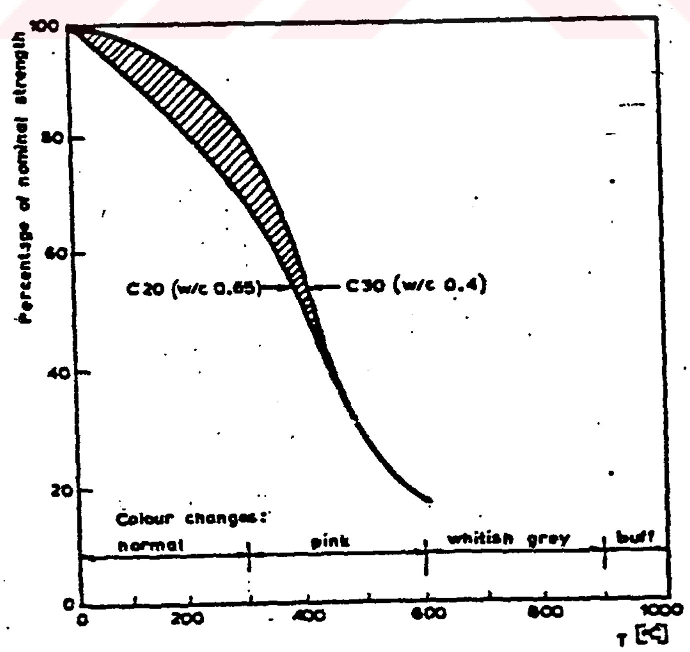


Fig. 4.13 Compressive strength of concrete after cooling (Concrete society technical report No.15, London 1978)

4.1.1.3 After Corrosion:

A number of reports have been published on the premature deterioration of reinforced concrete structures due to chloride corrosion of reinforcing steel. Splitting tensile stress due to volume expansion of corrosion product causes the longitudinal cracks in cover concrete along reinforcing steel. Okada, et. al. 1988 conclude that the yield strength of corroded beams with potential longitudinal cracks is scarcely reduced.[8]

Residual characteristics of constituent materials are needed for the subsequent numerical reevaluation of performance characteristics of building-elements.[2] Thus, for steel the following characteristics may be investigated:

1- Loss in cross-section (corrosion)
2- Loss in strength
3- Loss in ductility
4- Modifications of bond

Similarly, for concrete the following characteristics may be investigated:

1- Loss in cross-section (spalling)
2- Modification of strength (decreased in general, or increased under carbonation)
3- Modification of ductility (increase when micro-cracking has taken place)
4- Increase of creep.

4.1.2 Estimation of Residual Structural Characteristics:
(stiffness, strength, ductility)

4.1.2.1 General Method for Computation of Residual Characteristics:

The residual characteristics of R.C flexural members can be studied by computing their moment-curvature diagrams as based on the constitutive laws of materials. Theoretical moment-curvature diagrams for reinforced concrete sections with flexure and axial load can be derived analytically by means of various techniques, but the most used techniques is the following analytical models:[9]

1- The general fiber method.
2- The method of axially loaded substitute elements.

But the most convenient method for obtaining the moment-curvature diagram is the general fiber method, but the computational effort involved is rather large, so the digital computer is the most conveniegt for this method, where the section is considered to be composed of a number of horizontal element fibers, as shown in **Fig. 4.14**. The procedure consists of the following steps:

1- The section is divided into a number (n) of horizontal elements.
2- For a given top-srain and an assumed strain distribution, the average stress in each element can be computed from the assumed stress-strain material models.
3- In the case of cyclic loading, the stress-strain

history of each element has to be stored (into the computer) in order to calculate the stress corresponding to the given strain, employing the cyclic stress-strain material models

4- The forces acting on the elements are calculated from the stresses and the fiber's areas

5- The total internal forces are obtained by summing up the element forces over the section.

6- The equilibrium of the forces in the section must be checked. In general, this means an iteration with a new start in the assumed strain distribution and the above steps are repeated till the equilibrium equation:

$$\sum_{i=1}^{n_s} A_{si}.\sigma_{si}+\sum_{i=1}^{n_c} b_i.h_i.\sigma_{ci}+ N = 0$$

is satisfied, where N is the compressive axial force in the section.

7- The neutral axis depth x can thus be determined from the given top concrete strain, ϵ_{cr}, and the corresponding strain distribution, and the curvature 1/r can be computed from:

$$1/r = \epsilon_{cr} /x$$

and the moment from

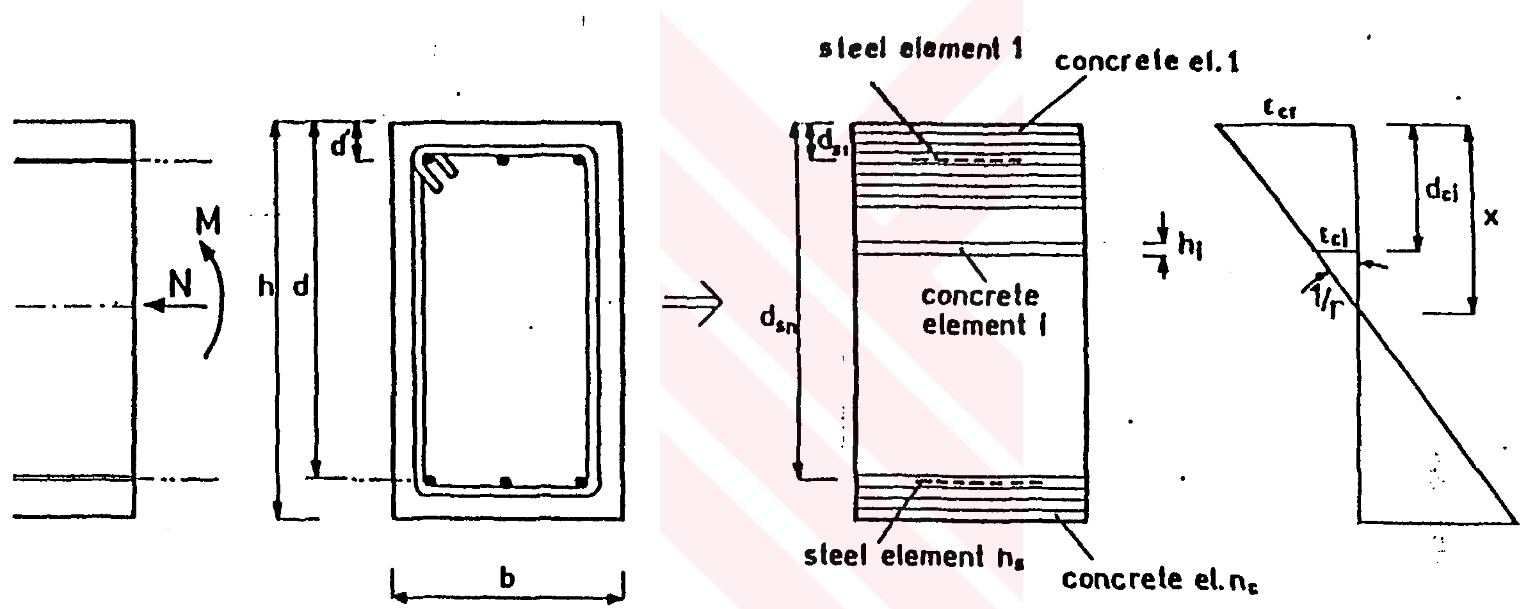
$$\sum_{i=1}^{n_s} A_{si}.\sigma_{si}.d_{si} + \sum_{i=1}^{n_c} b_i.h_i.\sigma_{ci}.d_{ci} + N.\ h/2$$

In this method (the general fiber method) the following assumptions have to be made:

1- The strain distribution across the section is usually linear (the section remains plane) in general even a non-linear distribution could be adopted.

2- Cracking of concrete occurs mainly perpendicular to the axis of bending.

3- Adequate shear reinforcement is present to suppress shear failure, so that the effect of shear force on the moment-curvature relations is not considered.



Average strain in element l : $\varepsilon_{cl} = \varepsilon_{cr} \cdot \dfrac{n \cdot \frac{x}{h} - l + 0{,}5}{n \cdot \frac{x}{h}}$

Fig. 4.14 General fiber method: section, discrete element and curvature

Chapter 5

DAMAGE OF REINFORCED CONCRETE BUILDING

5.1 Introduction:

Damage to engineering materials essentially results in a decrease of the free energy stored in the body with consequent degradation of the material stiffness. Most research in the field of damage assessment has dealt with the analysis of simple structural elements. As we know that any concrete structure is subjected to loading of increase intensity, it undergoes different phases of damage, from microcracking up to ultimate failure.

5.2 Fundamental Discussion of Damage and Failure of R.C. members:

Damage of a reinforced concrete member shall signify a specific degree of physical deterioration with precisely defined consequences regarding the member's capacity to resist further load. Similarly, "failure" of a member signifies a specific level of damage, which corresponds to a some-what arbitrarily defined residual capacity to resist further load. A damage index is usually defined as the damage value normalized with respect to the (arbitrarily defined) failure level so that a damage index value of unity correspond to failure.[10]

To understand the various factors that contribute to damage, it is necessary to review the behavior of plain concrete at different load levels. As concrete is subjected to stress, the stress concentrations near the microscopic flaws invariably lead to microcracks, which multiply and propagate either along the cement-aggregate interface or into the cement matrix itself. As the stress level exceeds about $0.5f'_c$, a more extensive and continuous macrocrack system begins to develop, which softens the material and give a strongly nonlinear stress-strain relationship this is explain by (Wittmann, Folker H., Fracture Mechanics of Concrete, Elsevier Science Publishers, Amsterdam,1983, 671 pp). Beyond approximately $0.75f'_c$, the growth rate of macrocracking accelerates until it becomes unstable, leading to failure.

The response of reinforced concrete to load is complicated by the complex interaction between steel and concrete. This is reflected by the numerous possible failure modes in flexure, shear, or bond. For dynamically applied cyclic loads, it is difficult to predict the failure mode for even "properly" detailed members. There are two reasons for this. First the so-called strain-rate effect influences the various failure modes to different degrees. (Thus, even a ductile steel member can experience a brittle fracture, because the resistance against sliding increases more rapidly with increasing loading rate than does the resistance to separation.) The second reason is that load reversals inflict different levels of damage in the various modes.[10]

5.3 Concrete Damage Models:

Numerous models have been proposed in the past to represent damage of structural members or entire structures. Several investigators have introduced energy indexes that are a function of a few selected parameters

The earliest references to damage are ductility based arising from considerations of two limit states of failure: monotonic loading and low cyclic fatigue. Several variations based on this concept have been proposed, including the recent "modified flexural damage ratio" of Roufaiel and Meyer (1987). Energy-based indices for damage prediction began to appear after Gosain et al. (1977). Among other recent models (Darwin and Nmai 1986; Stephens and Yao 1987), both ductility and dissipated hysteretic energy are given due consideration. Reviews of damage-index models Yao et al. (1986) and Chung et al. (1987) [10,11,12] show an increasing trend toward including aspects of both ductility and energy dissipation in damage assessment

The damage model proposed by Park, Young-ji, and Ang, Alfredo H-S., (1984), [11] is the only calibrated model based on actual observed damage of reinforced concrete buildings. In this model the structural damage is expressed as a linear combination of the damage caused by peak deformation and that contributed by hysteretic energy dissipation due to repeated cyclic loading:

$$D = \frac{\delta_m}{\delta_u} + \frac{\beta}{Q_y\ \delta_u} \int dE$$

in which D = damage index, an empirical measure of damage ($D \geq 1.0$ indicates total damage or collapse);
δ_m = the maximum response deformations
δ_u = ultimate deformation capacity under static loading
where for beams and columns:

$$\delta_u(\%) = 0.52\ l_s^{0.93}\ p^{-0.27}\ p_w^{0.48}\ n_0^{-0.48}\ (f'_c)^{-0.15}$$

and for shear walls:

$$\delta_u(\%) = 0.53 l_s^{1.23}\ p_h^{0.56} p_c^{-0.05} p_v^{-0.3} n_0^{0.09} (f'_c) 0.85$$

in which

l_s = shear span ratio = l/d ;
p = normalized steel ratio $=p_t f_y/f_c'$;
p_w= confinement ratio;
p_t= tension steel ratio;
n_o= normalized axial stress N/bdf_c';
p_h= horizontal reinforcement ratio(min. value 0.4%);
p_c= edge column reinforcement ratio(min. value0.2%)
p_v= vertical reinforcement ratio;
β = coefficient for cyclic loading effect (function of structural parameters)
$= (-0.447 + 0.073 i/d + 0.24 n_o + 0.314 p_t) \times 0.7 p_w$
Q_y = calculated yield strength;
dE = dissipated energy increment.

The above mentioned damage index (D) is represent damage of each component in the building. The

quantification of the local damage is not always sufficient for the description of the entire structure or parts of it. A strong damage index was therefore defined. Such an index is useful when analyzing weak-column strong-beam type buildings, where sudden shear drifts may trigger progressive collapse of the total structure. For the purpose of establishing the strong-level damage index, the local damage indexes D_i are averaged using a weighting factor based on the energy-absorbing capacity of elements:

$$D = \Sigma \quad D_i \; ; \quad E_i/\Sigma E_i$$

where is the energy weighting factor and E_i is the total energy absorbed by the component. In the case of strong-column weak-beam type buildings, it is necessary to extend the above concept to the entire structure. An overall damage index is obtained using the above equation for all the elements in the building. The overall damage index used has been calibrated (park et al. 1984) with observed damage data of nine reinforced concrete building that were moderately or severely damaged during the 1971 SAn Fernando earthquake and the 1978 Miyagiken-Oki earthquake in Japan. Table 1 corresponds to the calibrated index, which is meant only to serve as a qualitative indicator of the damage index used in the above mentioned calibration. As stated earlier, local damage of a more severe degree may be detected at the component or story level. The assessment of critical damage is then a matter of interpretation.

Table 5.1 Interpretation of damage (Park et al. 1984)

Degree of damage	Physical appearance	Damage index	State of building
Collapse	Partial or total collapse of building	>1.0	loss of building
Severe repair	Extensive crushing of concrete; buckled reinforcement	0.4-1.0	Beyond
Moderate	Extensive large cracks spalling of concrete in weaker elements	< 0.4	Repairable
Minor	Minor cracks throughout building; partial crushing of concrete in columns	-	-
Slight	Sporadic occurrence of cracks	-	-

The above mentioned damage index which is proposed by Park et al. (1984) remains the only calibrated model based on actual observed damage of reinforced concrete building, until more experimental and post-damage verification is made, this model is expected to serve as a useful indicator interpreting the overall damage sustained by the structure and its components.

Chapter 6

MATERIALS AND TECHNOLOGIES

Various technologies and materials commonly used in repair and/or strengthening works. Designers may not be familiar with some of these materials or techniques, more details (or even recommendations and specifications) for the materials and techniques can be found in many literature. In general, repair material should mainly meet the following specifications:

1- To be more durable than the initial material.
2- To protect steel (if possible by both raising the alkalinity of its surrounding and providing barriers to the ingress of harmful agents).
3- To be dimensionally stable (minimal shrinkage).
4- To achieve very good bond with cement and steel.

6.1 Cast-in-place Concrete:

6.1.1 Traditional Normal Cast-in-place Concrete:

Traditional normal cast-in-place concrete is often used in repair and strengthening works, but always the results are unsatisfactory, mainly because of the volume change shown by the cement-based concrete used, that impairs every possibility of a good contact between the new and the old element (build-up of stresses at the contact surface, decreasing of adhesion , separation and cracking). In order to improve the bond characteristics, and to minimize the shrinkage, it is recommended to do the following:[1,3]

1- It is recommended to use higher strength concrete ($f_{add} = f_{exist}+5MPa$) with low slumps.
2- Remove deteriorated or disintegrated concrete by means of a chisel or a scarifier.
3- Form more cavities in order to obtain a rough surface (local breaking of concrete) and uncover reinforcement.
4- Remove grease from concrete and rust from steel.
5- Remove dust by rinsing with water under pressure.
6- Saturate the old element for at least six hours before pouring the new concrete
7- Cure all exposed surface by wetting or by covering with wet burlaps for the required time (at least ten days).

6.1.2 Shrinkage Compensating or Expansive Concrete:

Concrete of this type is produced with expansive cements in place of normal cements, to obtain an appreciable volume increase which can just compensate the shrinkage of the mix. The expansive cements consist of the constituents of conventional portland cement with added or formed in the cement kiln expansive compounds; also, expansive admixtures (for example fine iron or aluminum powder) could be used with cement, water, sand and aggregates to produce these special type concretes; for many uses it is mandatory to use premixed expansive or shrinkage compensating grouts and mortars with a set of precisely known properties.

6.1.3 Polymer Modified Concrete:

Ploymer modified concrete is produced by replacing part of normal cement with certain polymers which are used as cementitious modifiers, or by replacing part of the mix water (or all of it) by polymers. The polymers, which are normally supplied as dispersions in water, act in several ways:

1- By functioning as water reducing plasticizers they can produce a concrete with better workability, lower water-cement ratio and lower shrinkage than normal concrete.
2- They can improve the bond between the old and new elements, securing almost monolithic behavior of the composite section after interventions.
3- They act as integral curing aides, reducing the need for effective curing (but not totally eliminating it) which may especially facilitate small jobs.
4- By introducing plastic links into the binding system of the concrete, they will improve the elasticity of the hardened concrete.
5- They can increase the resistance of the concrete to some chemical attacks.

However, it must be cautioned that such polymer modified concrete are bound to lose all additional properties when they will come under elevated temperatures and that the alkalinity and thus the resistance against carbonation will be much inferior to normal concrete (and therefore it is necessary to provide an adequate carbonation protection by a finishing coating).

6.1.4 Resin Concrete:

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

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

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

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

3- Although the direct bond of a resin compound on a clean and dry concrete surface is excellent, a resin concrete has generally no good direct bond on concrete, due to the fact that there can only be a punctual connection between the resin covered aggregates and the concrete; to assure a perfect bond , it is therefore necessary to apply a first coating of pure liquid resin onto the concrete surface.

4- Resin concrete will commonly have a much higher strength but also a lower modules of elasticity than normal concrete; the problems resulting from the different elasticity must be thoroughly considered for every construction.

6.2 Shotcrete (gunite):

When equipment and specially trained workmanship is available, the method of shotcreting is often desirable for strengthening reinforced concrete members. The widespread utilization of shotcrete for repair and/or strengthening works is based on three principal facts:

1- A very good bond can be obtained between the new shotcrete and the old concrete or masonry and the new reinforcement of shotcrete when properly applied on prepared surfaces.

2- The strength characteristics of shotcrete are high due to the high compaction energy and to a low water/cement ratio.

3- shotcrete can be sprayed on vertical, inclined and overhead surfaces with a minimum of formwork.

On the other hand the need of cracking prevention (due to shrinkage) is increased and so a minimum of skin reinforcement is always necessary and the curing of the thin concrete layers by repeated wetting of the surface is of extreme importance.

The surface of shotcrete should if possible be left as sprayed. If, however, another smooth surface required, a special thin layer is to be sprayed on the hardened shotcrete; this layer can then be reworked and finished to the required texture.

Surface preparation before shotcreting involves a thorough cleaning and removing deteriorated or disintegrated concrete by means of a chisel or a scarifier, sandblast, and also to open all the pores in the old concrete, and removing dust by rinsing with water under pressure, and saturating the old element for at least six hours before spraying the new concrete.

Shotcrete can be plain or fiber reinforced, with a minimum of skin reinforcement, as mentioned above. It can also be, as in the majority of cases, steel reinforced. All additional reinforcements have to be securately anchored on the surfaces of the existing element, in order not to tremble or to change their position during spraying.

Shotcrete is made from cement (the cement content should be 20% higher than for conventional concrete), aggregates of small size (5 to 10 mm) and a limited quantity of steel-fibers (usually less than 5% of the weight of the fresh concrete).

6.3 Resins:

Resins are conventionally used for injections, impregnations, glueing of thin steel sheets. These materials are two-component system; one component is a liquid resin (epoxy, polyester, polyurethane acrylic, etc.) while the second component is the hardener.

There is a large variety of available products with qualities and properties which may vary, depending on the kinds of components and their chemical structures, the mixing ratios, the quantity and type of filler and sand eventually added, etc. Therefore, the desired properties have to be precisely defined, in order to select the correct formulation of the synthetic mixture.

Resins used in the repair and strengthening should exhibit the following properties:

1- Adequate pot life, low hardening time and good workability.
2- Cure as far as possible independent of humidity and temperature.
3- Good tolerance for incorrect mixing.
4- Excellent bond characteristics and adhesion to concrete and steel and little or no reduction of adhesion as a function of time or of exposure to moisture.
5- Good, as possible, heat resistance
6- Low values of viscosity when the resins are used for injection and impregnation, and high values of viscosity when the resins are used for glueing.

7- Values of modulus of elasticity not very low, to avoid local reduction of stiffness of structural elements.

(i) <u>Injections and impregnations</u>:

Several different injection techniques may be used, such as low pressure injection (up to 1 MPa), high pressure injection (up to 20 MPa), or vacuum injection each injection technique requires a special resin adapted to the system. When the width of the cracks is very small (crack width 0.1 to 0.2mm), a pure resin is used, without any filler. In case of wider cracks, it is advisable to add a filler for reducing shrinkage, creep and thermic phenomena. Glass or quartz powder can be used as long the crack width is no more than 1.0 to 1.5 mm, and sand beyond this limit and up to 4.0 to 5.0 mm; the maximum dimension of grains must be not larger than 50% of minimum width of crack and no more than 1.0 mm in any case, using resin/filler ratio of about 1:1.

(ii) <u>Gluing of thin steel sheets</u>:

This strengthening method consists in glueing to an existing R.C element additional steel, in form of plates. This technique need special requirements such as, excellent bond between resin and concrete as well as between resin and steel, which demands special surface conditions and climatic conditions during the bonding phase.

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

6.4 Grouts:

Grouts are frequently used in repair and strengthening work to fill voids or to close the space between adjacent portions of masonry or concrete. Many types of grout are available. Conventional grout consists of cement, sand and water and is proportioned to provide a very fluid mix. grout of this type has excessive shrinkage characteristics due to the high volume of water in the mix.

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

Non-shrink grouts are available for use when it is desirable to fill a void without the normal shrinkage cracks. The dry ingredients for non-shrink grout comes premixed in sacks from the manufacturer and mixed with water.

Epoxy or resin grouts are also available for conditions when high shear force or positive bonding necessary across a void. Also many others types of grouts can be created using polymer products and other newer concrete products. The executing of grout which comes premixed from manufacturer must be mixed and used in strict according to the instructions of the manufacturer.

6.5 Additional Reinforcements:

Additional reinforcement are frequently used in repair and/or strengthening works, such as:

1- Glued or nailed thin steel plates (for flexural and/or shear strengthening).
2- Re-bar and/or welded fabrics for the same purposes
3- Additional stirrups in form of collars or bands, for shear strengthening of R.C. elements.

In order to assure the transfer of forces from the existing element to the new reinforcements, certain adequate measures have to be taken.

i) <u>Reinforcement:</u>
The electric welding method is the most common method to connect existing to additional reinforcements. New re-bars are welded to the "old" ones by large diameter intermediate bars, additional diagonal bars (shaped) or additional bent-down bars (shaped). According to the national codes, the techniques to be applied should be chosen, taking into consideration the weldability of the steel.

ii) <u>Collars or bands:</u>
There are two main techniques of shear strengthening of R.C elements by means of additional stirrups, tensioning and tightening

technique, bonding technique. The first technique is used for beams, columns an beam-columns joints, while the second one is used only for beams and columns. Collars made of the following form:

1- Collars made of narrow rather thin steel plates (e.g. plate No.30.4) installed around the element, secured in position (vertical, horizontal or diagonal) and then tensioned and tightened through bolts and nuts by means of a wrench. In an alternative technique use is made of round bars (with a diameter more than 16mm) of high strength.
2- Collars made of the same plates (or mild steel round bars With a diameter more than 10 mm) welded on angle bars positioned on the corners of the R.C. elements. The angle bars have to be clamped and pressed against the existing element or the collars have to be preheated (to 200-400°C) and then welded on the angle bars.
3- Mild steel round bars (with a diameter of 5 or 6 mm) heat tensioned and then hammered around the element, to form a rectangular spiral, angle bars can be positioned on the corners and the spiral can be welded on them, especially at the ends.
4- Glued (or nailed) wide and thin steel sheets (e.g. plate No. 50 or 100) around the element.
5- Wide and thin steel sheets heat tensioned, welded in-between them, and finally bonded by injections.
6- Tensioned packing-bands around the beam's or column's damaged area, subsequently secured with pressed clips.

Tow general remarks have to be mentioned, as far as the additional stirrups are concerned:[1,2]

a) The possibility of arranging these additional reinforcements on the surface of R.C. element (external collars, spirals or bands) or on the core of the element, removing all concrete cover of the reinforcements.

b) The necessity of covering all these strengthening collars and bands by a layer of shotcrete or of cement mortar with an incorporated wire trellis.

This protection is of great importance in case of bonded reinforcements, as mentioned in the previous technology of resins.

6.6 Incorporated Steel Profiles:

In this technique use is made of a light steel structure attached on a damaged R.C. element, (beam and mainly, column) for flexural and shear strengthening. The technique comprises four longitudinal angle profiles on the corners of the element temporary clamped and pressed against the existing element (by means of screw clamps or transverse L-angle profiles in pairs and tightened firm to each other through high-strength bolts), and transversal steel strips or round bars as stirrups, welded on the angle bars. The longitudinal angle bars can be fitted on non-shrinking cement g rout, in order to achieve a good contact. Finally, the whole arrangement is covered by concrete jackets (cast-in-place concrete or shotcrete) with additional reinforcements or a minimum of skin reinforcement (mainly welded fabrics), or by a layer of shotcrete or of cement mortar with an incorporated wire trellis.

6.7 Infilling and Bracing Techniques:

The technique of infilling new shear walls are mainly used for the strengthening of a building as a whole (damage and/or weak R.C, frames). New bearing elements infilling and/or bracing of existing frame bays) are arranged in order to improve the overall behavior of a building (i.e strength, stiffness and ductility), when subjected to shear deformations (lateral forces, differential settlements, etc.). There are several methods and technologies for infilling and/or bracing existing R.C frames, among which the most commonly used are the following:

1- Cast-in-place R.C. infilling walls connected to the frame (or un-connected). The infill element can be connected with the frame along its perimeter or only along the beams

2- Precast R.C. infilling panels connected to the frame; these prefabricated panels can fill the whole bay or part of it , the precast panels may be secured in position and connected to the frame by means of steel anchors (expansion or adhesive type). 3- Masonry (brick or concrete block) partition-infilling walls, these masonry walls can be reinforced or not.

4- R.C. wing walls, cast-in-place or precast. The wing walls are connected to the R.C. column as well as to the R.C. beams, at their top and their bottom, by means of steel anchors.

5- Arrangement of various types of steel bracings (X-type, K-type, Knee-type, etc.), steel trusses and steel frames, connected to the frames.

Fasteners are used at the joints between the added new elements and the existing structural members in some of the mentioned above technique In the past only mechanical anchors were accepted as the jointing device in the strengthening design, now a days bond anchors are accepted also as shear connectors.

Chapter 7

REPAIR AND STRENGTHENING TECHNIQUES

7.1 Introduction:

Repair and strengthening techniques are naturally as old as construction techniques itself, but these techniques have started to receive researchers attention only recently. Many basic concepts related to repair and strengthening has been confirmed by experimental works. In the structural sense repair can be defined as any operation to recover the structural performance of element or structure with damage to the original performance, and strengthening can be defined as any operation to increase the structural performance of a element or structure over the original performance. Reliable experimental data available on the repair/strengthening of reinforce concrete structures and the techniques used will be discussed in this section.

7.2 Columns:

The aim of repair/strengthening is to improve the seismic performance of the buildings. Preventing shear failure of the columns, improving ductility, and rearrangement of the columns stiffness and making them unify as possible, or the increasing flexure capacity of columns may be aimed to improve the seismic performance of a building. Column flexural strength increase with the enlargement of the concrete area and adding new longitudinal reinforcement. Shear strength and especially ductility, is improved by better confinement with close transverse reinforcement, ties or steel strips, or by winding by carbon fiber wires or steel wire strand. Unify column stiffness by rearrangement, like separating columns from spandrel walls, improves the compatible behavior of all columns of the structure.[1,2]

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

7.2.1 Local Repairs of Columns:

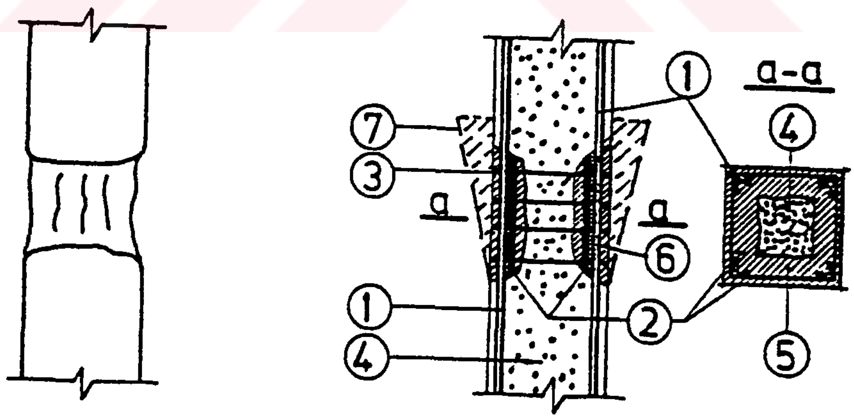
Several techniques have been developed in recent years for repairing damage concrete columns, and the repairing techniques depend on the evaluation of the damage. These techniques of repairing can be schematically subdivided them into the following categories:

i) Injections and impregnation: with epoxy used to fill cracks of limited amplitude (up to 6mm), and without damage of concrete or reinforcement. The general procedures involved in epoxy injection and impregnation are as follows:[13]

a) Injection techniques generally consists of drilling holes atclose intervals along the cracks, in some cases installing entry ports, and injecting the epoxy under pressure. For massive structures, an alternative procedure consists of drilling a series of holes [usually, (22mm) in diameter] that intercepts the crack at a number of locations. Typically, holes are spaced at (1.5m) intervals. More details about this technique is described in ACI 503R-80 "Use of Epoxy Compounds with Concrete". With the exception of certain specialized epoxies, this technique is not applicable if the cracks are actively leaking and cannot be dried out.

b) Vacuum impregnation is the second method to introduce epoxy into damage concrete. This method has been used in Great Britain to repair range of deteriorated structures, including bridge piers, sculptures, and pavements. The vacuum impregnation procedure, the loose chunks of material are integrated into the repaired regions. The repair region is subjected to a partial vacuum, epoxy is then drawn through the system until it is completely submersed and, just before the epoxy sets, the vacuum is released. The pressure difference between the vacuum and the evacuated system forces the epoxy into the remaining cracks and void spaces. Catherine Wolfgram French, Gregory A. Thorp, and Wen-Jen Tsai (1990) show experimentally the effectiveness of epoxy techniques to repair moderate earthquake damage and conclude that, the vacuum impregnation technique may have an advantage in the repair of large regions of damage and offshoot cracks.

ii) <u>Reconstruction with concrete</u>: used to repair heavily cracks or spalled concrete portions, and/or to increase the section of a structural element. it is usually associated with substitution and/or addition of reinforcement. This techniques can be applied both to columns and beams. prior to repair, the damaged concrete must be demolished and removed, and the exposed surface cleaned of any dust or loose material. Fig. 7.1 show the method of casting new concrete. Additivated cement concrete or resin concrete can be used, but the last is the most expansive, is used when only limited volume are necessary; the first requires a glue to ensure adhesion to the old surface, and is subject to cracking due to shrinkage. Shotcrete application can be viewed as a special case of this technique with improved adhesion to the old concrete.

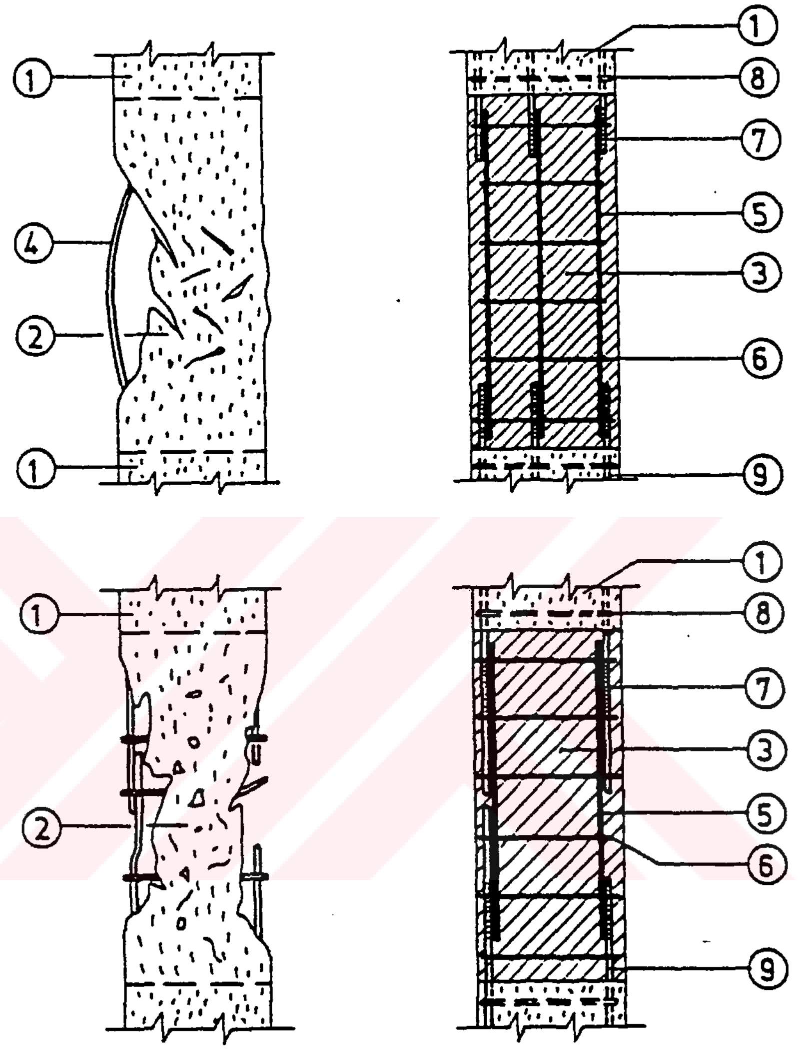


1 - existing reinforcement
2 - added new reinforcement
3 - added new ties
4 - existing concrete
5 - new concrete
6 - welding
7 - temporary castform

Fig. 7.1 Repair of damaged column

iii) <u>Repair or addition of metal reinforcement</u>: when the longitudinal reinforcement is buckled, the ties are ruptured and the concrete is crushed, total removed and replacement of the damaged parts must be carried out, Fig. 7.2 show the method of buckled reinforcement repairing. Non-shrinkage concrete or concrete with low shrinkage properties should be placed. Special attention must be paid to achieving good bond between the new and the existing concrete.

iv) <u>Incorporated steel profiles</u>: is mad of light steel profiles attached on damaged or inadequate R.C. elements (beam and mainly column) for repair and/or strengthening. This technique as shown in Fig. 7.3 , used for repairing damaged R.C. column with degree of damage from light to medium. The technique comprises four longitudinal angle profiles attached on the corners of the column and temporary clamped and pressed against the existing column (by means of special screw clamps or transverse pairs of L-shaped steel profiles tightened firm to each other through long high-strength bolts), and transversal steel strips (wide and thin ones) or round bars as stirrups closely spaced and thoroughly welded (full strength welds) on the angle bars.



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

Fig. 7.2 Method of repairing damaged column with buckled reinforcement

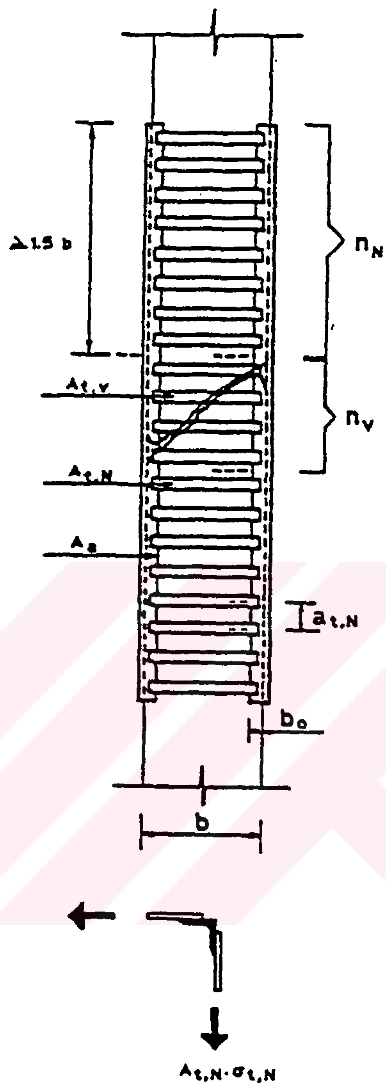


Fig. 7.3 Construction technique for strengthening by lightsteel profils

7.2.2 Techniques for Strengthening Columns:

The techniques to jacket the existing column by the concrete and welded wire fabric or by the steel plates are very popular, and they have many experimental back data to refer the details such as:

1- Japan Building Disaster Prevention Association: GUIDELINE FOR SEISMIC RETROFITING DESIGN OF EXISTING REINFORCED CONCRETE BUILDING, 1977, revised in 1990.
2- C.E.B. Bull. 162: ASSESSMENT OF CONCRETE STRUCTURES AND DESIGN PROCEDURES FOR UPGRADING (redesign).

The basic targets for strengthening column techniques are, to improve ductility, to adjust stiffness distribution of columns, and to increase flexural capacity. One important thing is the providing the gaps at both of the column ends to keep the original column section and to keep the original flexural strength. So that the jacketing materials to become effective only for the preventing shear failure and the confining concrete.

7.2.2.1 Strengthening to Improve Ductility:

The following techniques are available to improve the ductility of a column:

1- Jacketing by welded wire fabric and cover concrete
2- Jacketing by welded closed hoops and cover concrete.
3- Jacketing by steel plates and injecting grout mortar.
4- Jacketing by steel strips and cover concrete.
5- Winding by carbon fiber wires or steel wire strand

Fig. 7.4 show the Japanese techniques for strengthening columns for improving ductility.

According to the Japanese techniques "GUIDELINE FOR RETROFIT TECHNIQUES OF EXISTING REINFORCED CONCRETE BUILDINGS"[14] the following detail should be require in each technique:

(a) Providing the gap whose clearance should be approximately 30 mm. at the top and bottom of a column to remain the original column section and to keep the original flexure strength.

(b) Chipping the original cover concrete of a column to fit the new cover concrete with the original concrete.

(c) The compressive strength of cover concrete or mortar should be more than 21 MPa and more than the strength of original concrete.

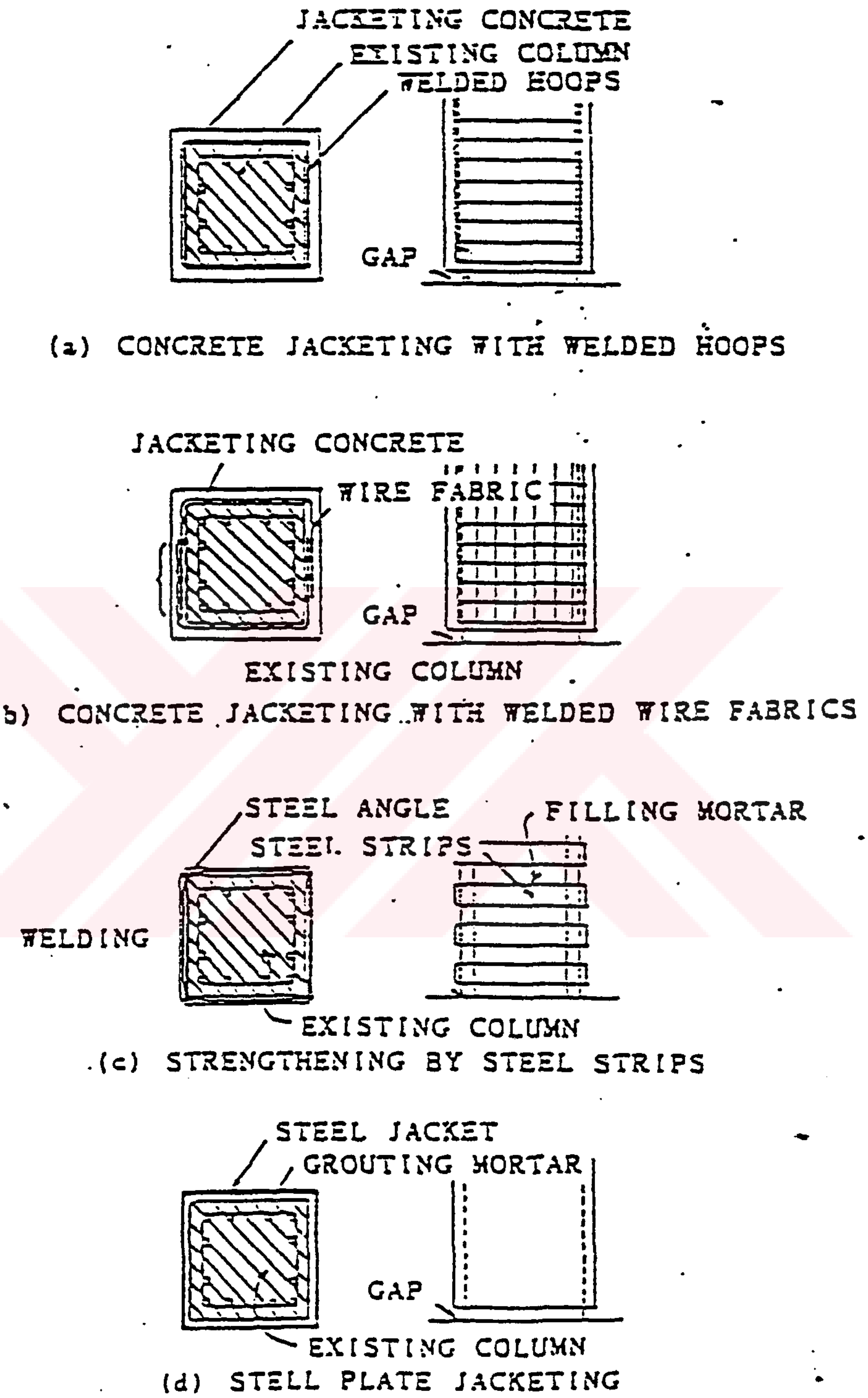
(d) The length for lap-joint of welded wire fabric should be more than 200 mm. and more than the length of transverse wire's pitch plus 100 mm.

(e) D10 bars should be used for hoops, and the spacing of hoops should be less than 100 mm. for the jacketing by welded closed hoops. Both ends of a hoop should be welded each other, and the lap weld length should be more than ten times of the hoop diameter.

(f) The thickness of cover concrete for welded wire fabric or hoops should be more than 60 mm.

(g) The thickness of steel plates or steel strips should be more than 4.5 mm

(h) The width and pitch of steel strips may be approximately 100 mm.



(a) CONCRETE JACKETING WITH WELDED HOOPS

(b) CONCRETE JACKETING WITH WELDED WIRE FABRICS

(c) STRENGTHENING BY STEEL STRIPS

(d) STELL PLATE JACKETING

Fig. 7.4 Techniques for strengthening columns to improve ductility

7.2.2.2 Strengthening to Adjust Stiffness Distribution of Columns:

The techniques, in which the non-structural spandrel walls in frames are demolished and removed or the effective gaps are provided in the joint region between non-structural walls and columns, are available to adjust the stiffness distribution of existing columns.

according to the Japanese techniques mentioned above[14] the following details should be required:

(a) If the shear failure will be assumed for the column after removal of non-structural spandrel walls, the column should be strengthened by some jacketing technique.

(b) The clearance of gap which is provided at the joint of non-structural spandrel walls should be more than 30 mm. and the non-structural spandrel walls with the gaps should be safe against the overturning.

(c) If the thickness of non-structural spandrel wall be remained partially in the gap, the thickness should be less than 50 mm.

(d) When the thickness of non-structural spandrel wall will be reduced to provide the gap, the existing rebars in the gap should not be cut off.

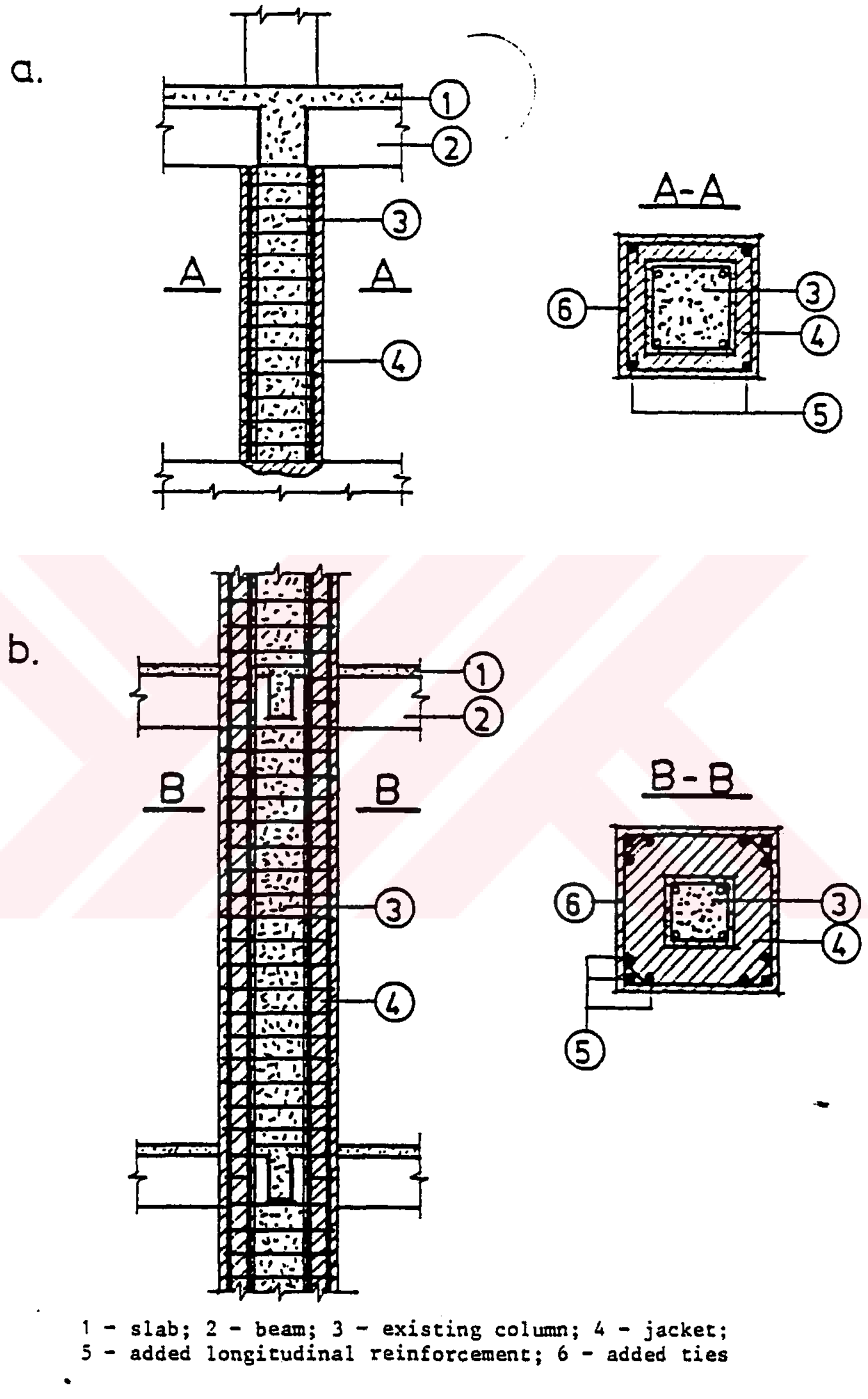
7.2.2.3 Strengthening to Increase Flexural Capacity:

The available technique to increase flexural capacity of columns is the jacketing by concrete with additional longitudinal and transverse reinforcement. Adequate column flexural strength can be achieved by passing the new longitudinal reinforcement through holes drilled in the slab and placing new concrete in the beam-column joint region (Fig. 7.5). The added longitudinal reinforcement can be connected to existing reinforcement by welding of bent bars as shown Fig. 7.6.a this jacket type is applied for large column cross sections where the middle reinforcement cannot be confined by new ties. If there is available space around the column, jacketing with ties by concentration of the newly-added longitudinal reinforcement at the corners of the cross section is the best for confinement of all longitudinal bars (Fig. 7.5.b). This type of jacket should be of sufficient thickness with closely spaced ties to provide confinement. With the new longitudinal reinforcement passing through holes drilled in the slab, this procedure provides a continuous connection of the jackets to the upper story and lower story columns. The following details should be required to increase the flexural capacity of a column by strengthening:

(a) The new tension reinforcement in a column should be developed beyond the top and bottom slabs and the anchorage of tension reinforcement should be provided enough.

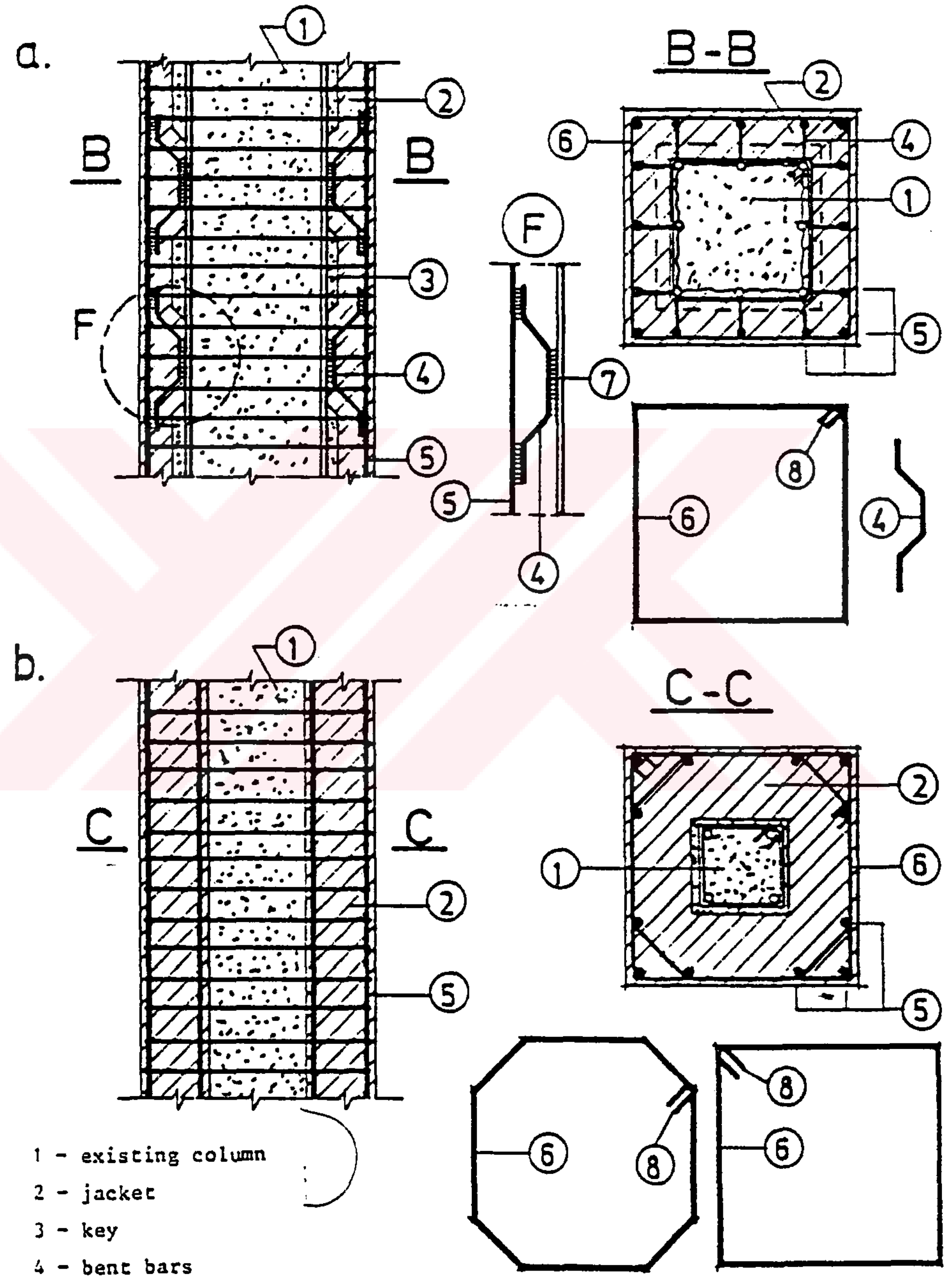
(b) The chipping for the original concrete surface of a column should be required to fit enough the joint concrete between the original and the new concrete.

(c) The shear reinforcement should be provided enough to prevent shear failure of a column after strengthening.



1 - slab; 2 - beam; 3 - existing column; 4 - jacket;
5 - added longitudinal reinforcement; 6 - added ties

Fig. 7.5 Techniques for strengthening columns to increase flexural capacity



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

Fig. 7.6 Construction techniqes for connection new reinforcement to the old reinforcement

7.3 Beams:

Many methods of repair and/or strengthening of beams are proved by experimented works, for providing adequate strength and stiffness of damage or undamaged reinforced concrete beams. It is very important that the repair and/or strengthening procedure chosen provide proper strength and stiffness of the beams in relation to adjacent columns in order to avoid creating structures of the "strong girder-weak column" type which tend to force seismic hinging and distress into the column, which must also support major gravity loads.

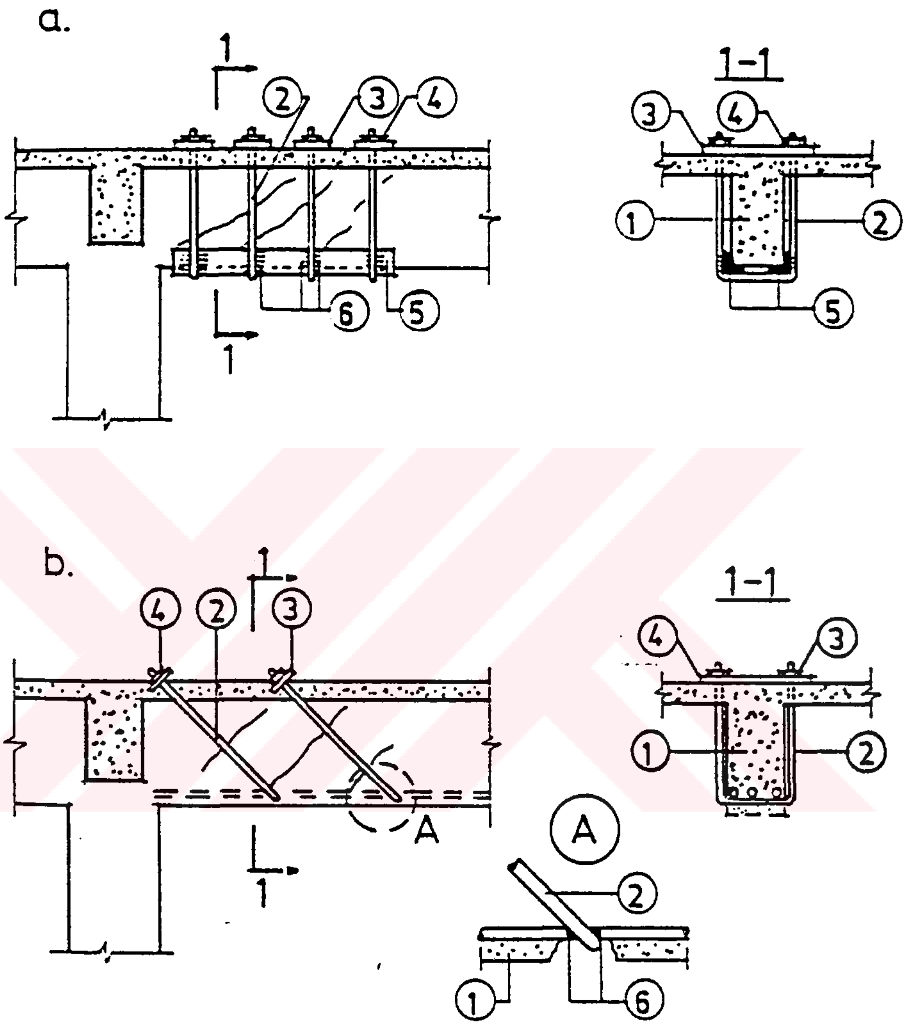
7.3.1 Local Repairs of Beams:

Several techniques have been developed in recent years for repairs damaged reinforced concrete beams. Repairing techniques of damaged reinforced concrete beams can be subdivided into the following:

i) Injection: are applied for repair of damaged beams with slight cracks only. Epoxy or cement grout injections are made according to the recommendations in section 7.1.2, and similar to such repair for columns.

ii) Reconstruction with concrete: used to repair heavily cracked or spalled concrete, crushed concrete, deterioration of bond and rupture of reinforcement. Before the removal of damaged concrete or rupture reinforcement, the damaged beam must be temporarily supported. The reconstruction procedure of beam is similar to that of columns. More attention must be paid to compacting the new concrete under existing beams or slabs which is extremely difficult if placement access is not provided from the top surface of the beam.

iii) Repairing gravity load capacity of beams: steel rods can be used for improving the shear resistance of damaged or undamaged beams. It can be performed by vertical external clamps (Fig 7.7.a) or by diagonal ones (Fig. 7.7.b). The clamps consist of round rods with threads at the end which are tightened with nuts. At the beam, bottom vertical clamps are fixed on angle profiles, but diagonal clamps are welded to longitudinal reinforcement to resist the longitudinal component of force. If load reversal are anticipated, four-sided jacketing is the preferred method of strengthening.



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

Fig. 7.7 Construction techniques for repairing gravity load capacity of beams

7.3.2 Techniques For Strengthening Beams:

The aim of strengthening of beams is to provide adequate strength and stiffness of damaged or undamaged beams which are deficient to resist gravity and seismic loads. More attention must be paid to avoid creating structures of the "strong girder-weak column" type as mentioned above for repairing techniques. It is possible to subdivide the strengthening techniques of beams into the following:

1- Strengthening to increase flexural capacity of beams.
2- Strengthening to increase shearing capacity.

7.3.2.1 Strengthening to Increase Flexural Strength of Beams:

The available techniques to increase flexural strength of a beam are reinforced concrete jacketing and glued thin steel sheets.

i) Reinforced concrete jacketing: can be performed by adding concrete on one, three or four sides of a beam. This method can be achieved by casting a new concrete layer with new longitudinal reinforcement after chipping the concrete cover. To ensure bonding between the old and new concrete layer, either Z- or U-shaped connectors welded to the existing steel are used. An irregular shaped concrete surface by the roughening of the concrete combined with anchoring by welded stirrups provides good shear and tensile connection of the jacket to the existing beam. Reliable anchorage of longitudinal reinforcement bars in join areas by sufficient development length or reinforcement welding to steel anchor plates or profiles is of great importance. Adequate shear and ductility improvement should be provided by additional stirrup distributed on all sides of the existing beams. The legs of the stirrup should penetrate through drilled holes in the slab in the top part of the jacket, where they must be sufficiently anchored.

One-sided jackets Fig. 7.8 or adding strength only to the beam soffit should be used only where it is necessary to increase the flexural strength in the mid-span of a beam, and this technique must be achieved according to the recommendation mentioned above.

Four-sided jacketing Fig. 7.9 or encasement of a beam adds considerable flexural and shear strength because of the increase of the reinforcement area and concrete cross section dimensions (depth and width). The additional longitudinal reinforcement should be connected with the existing reinforcement by diagonally welded bent bars or small steel plates. The stirrups pass through holes drilled in the slab and surround the whole beam. These holes can be used to place the concrete in the part of the jacket beneath the slab. Additional reinforcement for negative bending moments must be added over the slab surface in the beam region and outside of the existing column. special attention must be paid to the anchorage of the longitudinal bars in the joint region of the column jacket.

Jacketing on three sides of the beam can also be installed beneath the soffit of the slab, as shown in Fig. 7.10. Shotcrete is the most feasible method of installing this type of jacket and its primary weakness is the anchorage of the new stirrups at the sides of the jacket. The effectiveness of the stirrups anchorage depends on dynamic strength of the power driven nails and the stiffness of the strand to provide effective anchorage for new stirrups.

Reinforced concrete jacketing should conform to the following provisions:

(a) The strength of the new materials should not be less than those of the existing beam.

(b) The thickness of the jacket should not be less than 40 mm for shotcrete application or 80 mm for cast-in-situ concrete.

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

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

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

ii) <u>Glued thin steel sheets</u>: Epoxy-bonded steel plate has been used effectively in Europe, South Africa, and Japan to increase the load-carring capacity of existing concrete bridge girders. This strengthening method has been found by practitioners to be economical and efficient to apply. The principals of this technique are fairly simple. Steel plates are epoxy-bonded to the tension side , increasing the strength and stiffness of the beam. A shortcoming of this technique, however, is the danger of corrosion at the epoxy-steel interface, which adversely affects the bond strength. The following requirements must be considered and followed such as:

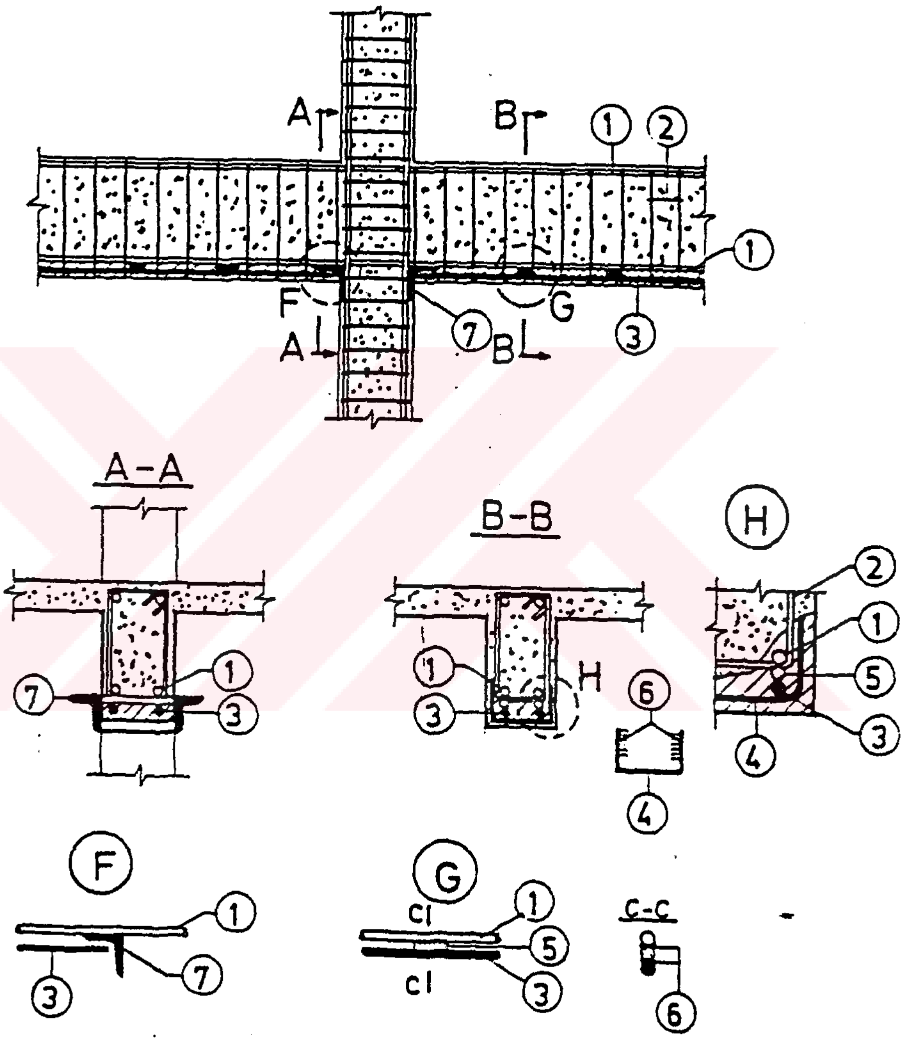
(a) The concrete must be cleaned from all loose bits and hardened laitance (as well as grease and oil) by sandblasting, pneumatic bush-hammers and wire brushes; careful smoothing and removal of dust.

(b) The steel plates must be dipped briefly into acid solutions (for removing of scales and oxides) and then applying trichloroethane, a grease and oil dissolving cleaning agent.

(c) The thickness of the resin layer should not exceed 1.5 mm, and steel plates thickness should not exceed 3.0 mm (for the plate, in case special anchorage systems are used the thickness could be higher but no more than 10 mm).

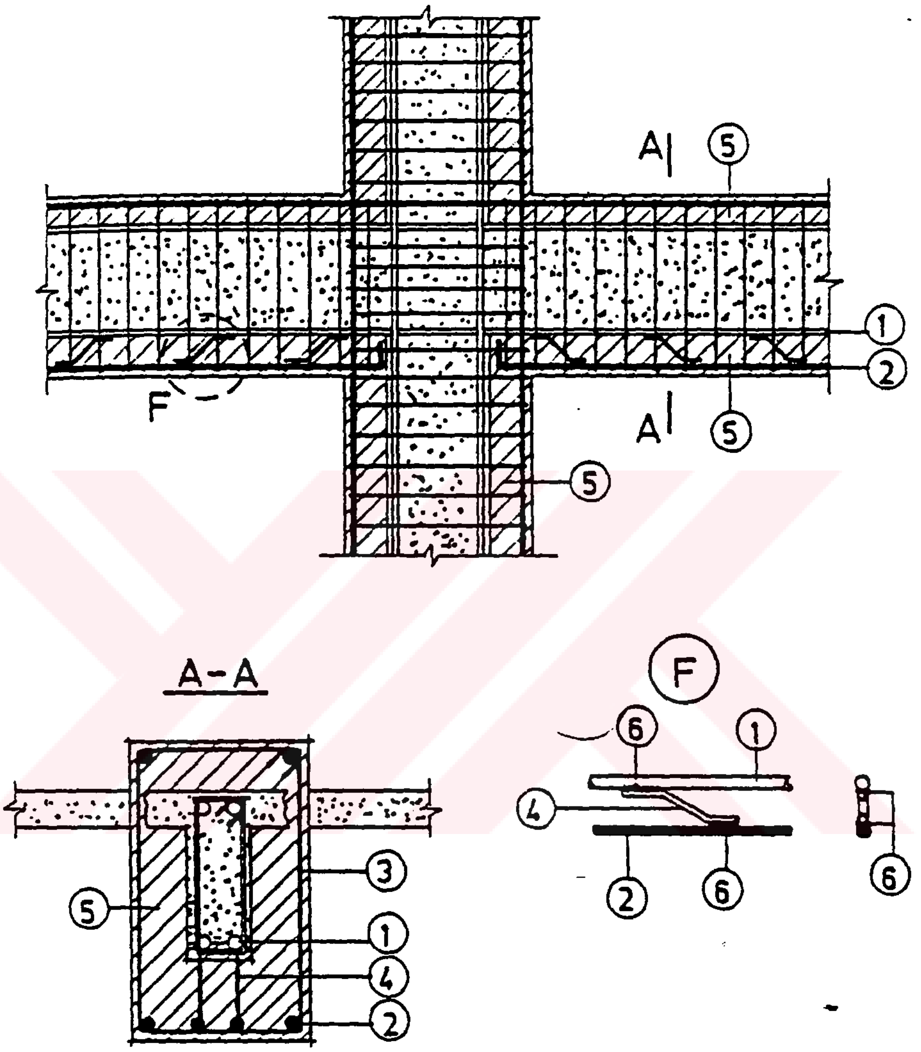
(d) Clamping up of all bonded steel sheets on concrete elements by screw clamps, using light but uniform pressure for at least 24 hours (depending mainly on resin formulations and on ambient temperature).

(e) Finally, the strengthened part of the concrete element has to be protected against temperature changes and especially against fire (by means of a layer of rich cement mortar with an incorporated wire trellis, or by more appropriate means).



1 - existing reinforcement; 2 - existing stirrups; 3 - added longitudinal 4 - added stirrups; 5 - welded connecting bar; 6 - welding; 7 - collar of angle profiles

Fig. 7.8 Construction techniques for strengthening to improve flexural capacity of beams by jackreting on beam soffit



1 - existing reinforcement; 2 - added longitudinal reinforcement;
3 - added stirrups; 4 - welded connecting bar; 5 - concrete jacket;
6 - welding

Fig. 7.9 Constructipn techniques for streninthening by four-sided jacketing of beame

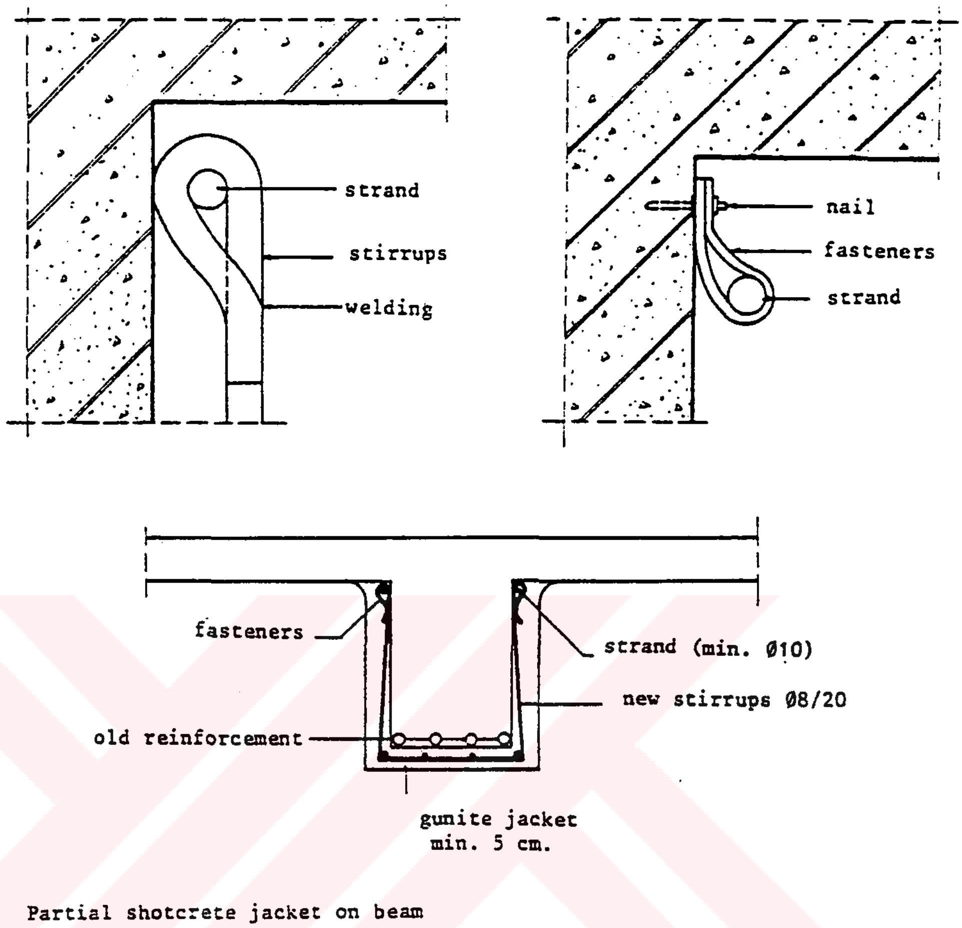


Fig. 7.10 Construction technique for strengthening by three sides jacketing of beams

7.3.2.2 Strengthening to Increase Shearing Capacity of Beams:

The available techniques to increase shearing capacity of a beam are reinforced concrete jacketing and glued thin steel sheets.[2]

i) Reinforced concrete jacketing: can be performed by adding concrete on three, or four sides of a beam with longitudinal reinforcement and stirrups. The same requirement mentioned in section 7.2.2.1 paragraph (i) be thoroughly considered. Additional requirements (for minimum percentages of additional reinforcements, minimum additional thickness, etc.) are shown in Fig. 7.11, for the usual case of combined shear and flexural strengthening attention is drawn on the need of careful anchoring of all additional shear reinforcement (stirrups, diagonal bars) of the R.C. jackets.

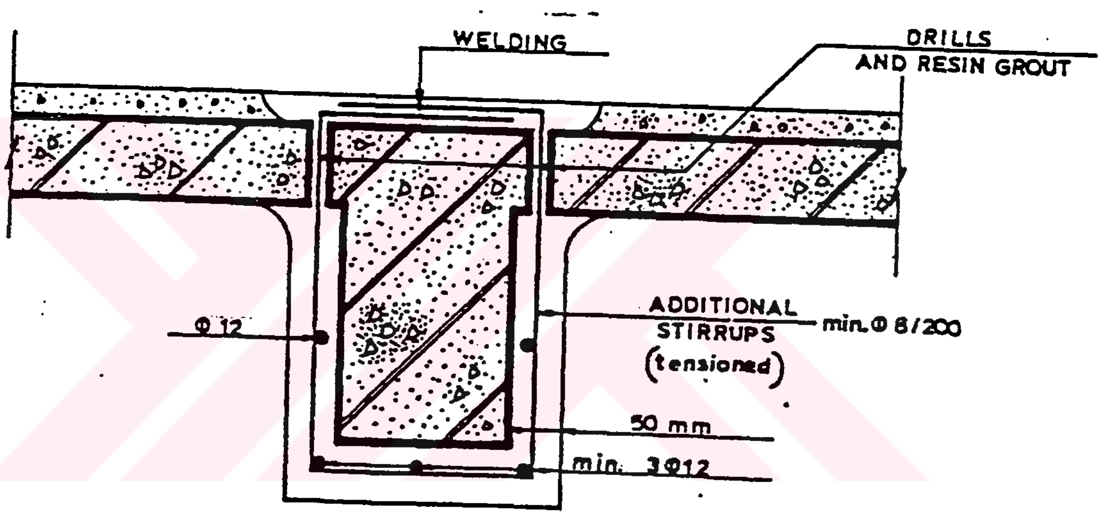


Fig. 7.11 Strengthening by means of R.C. jackets and additional stirrups

ii) <u>Glued thin steel sheets</u>: can be performed by gluing thin steel sheet on the two sides of a beam by epoxy resin in the zone of maximum shear force. The same requirements mentioned in section 7.2.2.1 paragraph (ii) (in regard with thicknesses of sheets and bond layers, surface preparation, protection against fire, etc.) have to be thoroughly considered and followed. Fig. 7.12 show the case of combined flexural and shear strengthening by means of glued thin sheets for beams.

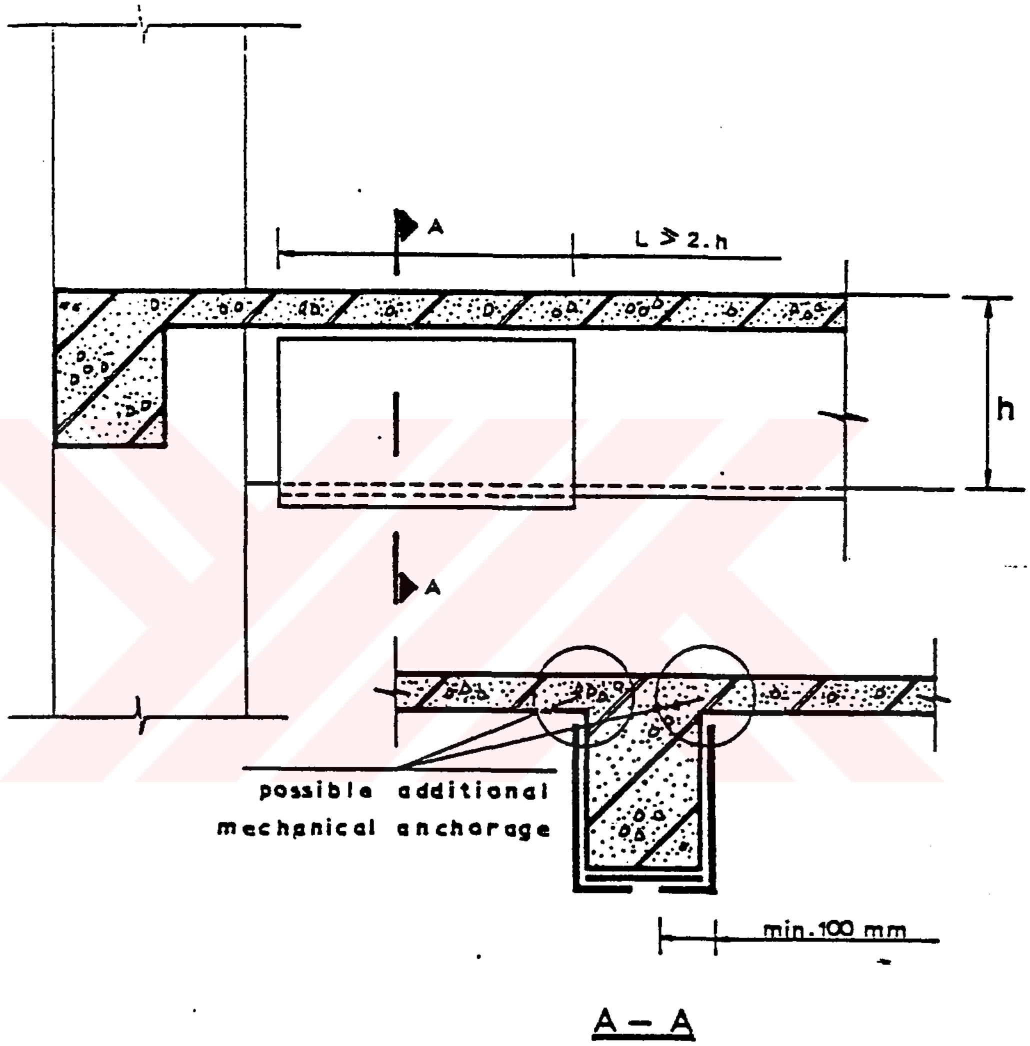


Fig. 7.12 Combined strengthening by means of glued thin stell sheets

7.4 Beam-Column Joints:

Joints are often the weakest links in structural system, and they are the most difficult to strengthen because of the great number of elements assembled in them, and also due to the high stresses subjected to them especially during an earthquake. Under earthquake loading joints suffer shear and/or bond (anchorage) failures. Severe earthquake excitations can produce plastic hinges in the beams at the column faces. As a result, cracks develop throughout the overall beam depth. Bond deterioration near the face of the column causes propagation of beam reinforcement yielding into the joint and a shortening of the bar length available for force transfer by bond causing horizontal bar slippage in the joint.

7.4.1 Local Repairs:

Epoxy injections can be applied for repair of damaged joints with slight to moderate cracks without damaged concrete or bent or failed reinforcement. restoration of bond between reinforcement and concrete by injections is inadequate and unreliable. Epoxy injections are to be performed according to the recommendations in section 6.3. Removal and replacement should be applied in cases of crushed concrete, deteriorated bond or ruptured reinforcement. Before the removal, the damaged structure must be temporarily supported. The replacement procedure is similar to that of columns.

7.4.2 Techniques for Strengthening Joints:

The available techniques for strengthening joints are reinforced concrete jacketing and steel plate reinforcement.

7.4.2.1 Reinforced Concrete Jacketing:

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

In cases of damaged or poorly detailed joints, the reinforced concrete jacket can be located in the joint area only **Fig. 7.13**. The jacket encloses the column, the beam and the joint at all faces. Horizontal ties provide the necessary shear strength of the joint. Transversal vertical stirrups are connected with the horizontal ties. Note that the jacket projects above the top of the structural slab.

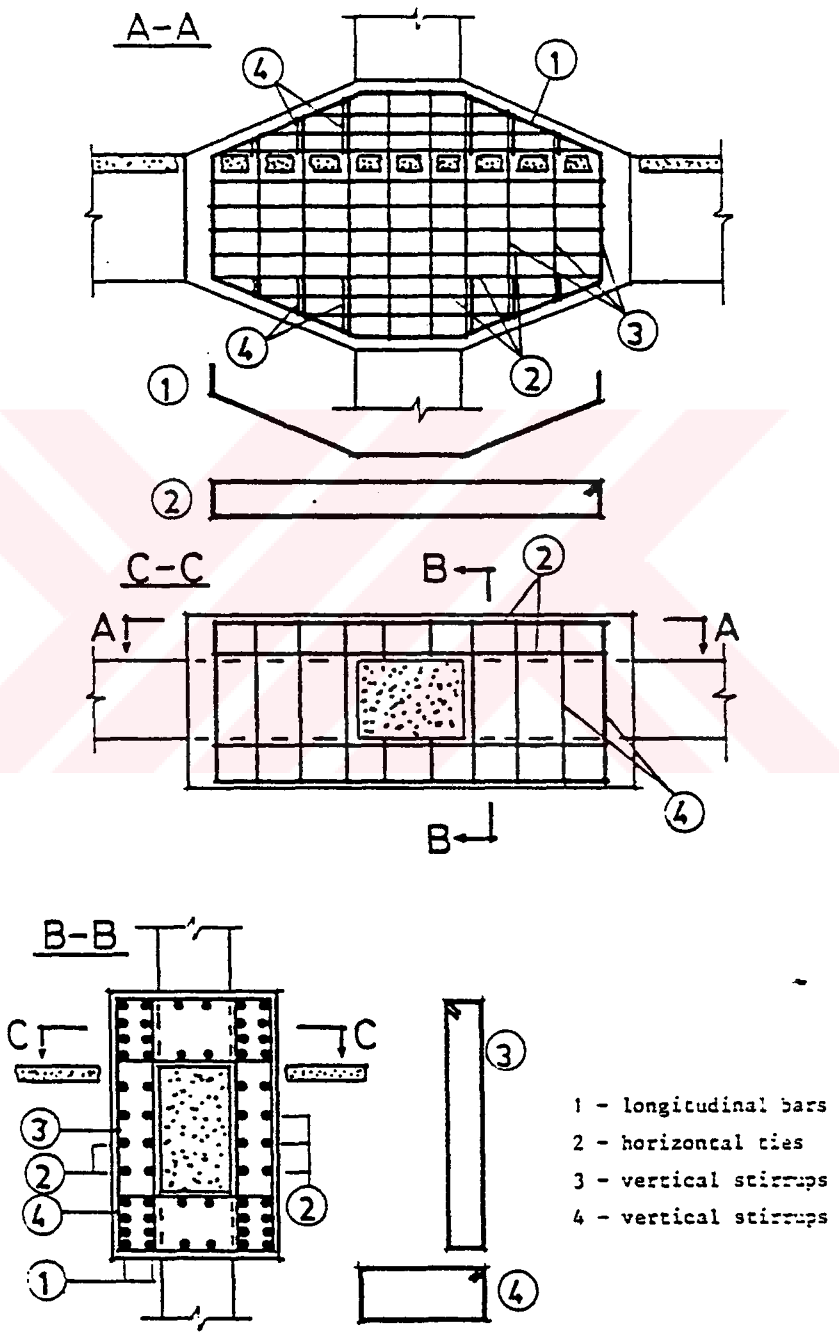


Fig. 7.13 Tecnique for strengthening column-beam joint by concrete jacket with additional reinforcement

For heavily damaged structures interconnected jacketing of the beam, columns and joints is desirable Fig.7.14. The column and beam additional reinforcement passes through the joint area. Additional horizontal and vertical ties in the joint area must be placed for providing adequate joint shear strength. Above and under the cross beam, horizontal column ties in the joint area must be closely spaced and spliced by overlapping. In case of deep cross beams, a connection with some horizontal ties passing through the beam web (i.e. through drilled holes) is advisable.

Depending on the forces acting in the area of an exterior joint Fig. 7.15 appropriate reinforcement should be provide in the joint jacket. Additional horizontal and vertical bars from the column and beam penetrate into the joint area. Closely spaced welded wire fabrics or reinforcing bars are placed at both of the joint faces. Welded wire fabrics in the joint area should be connected by welding to the original reinforcement or by attachment to the concrete with epoxy anchored dowels or power-driven nails. Additional stirrups or ties in the beam and column close to the joint improves the performance of this connection.

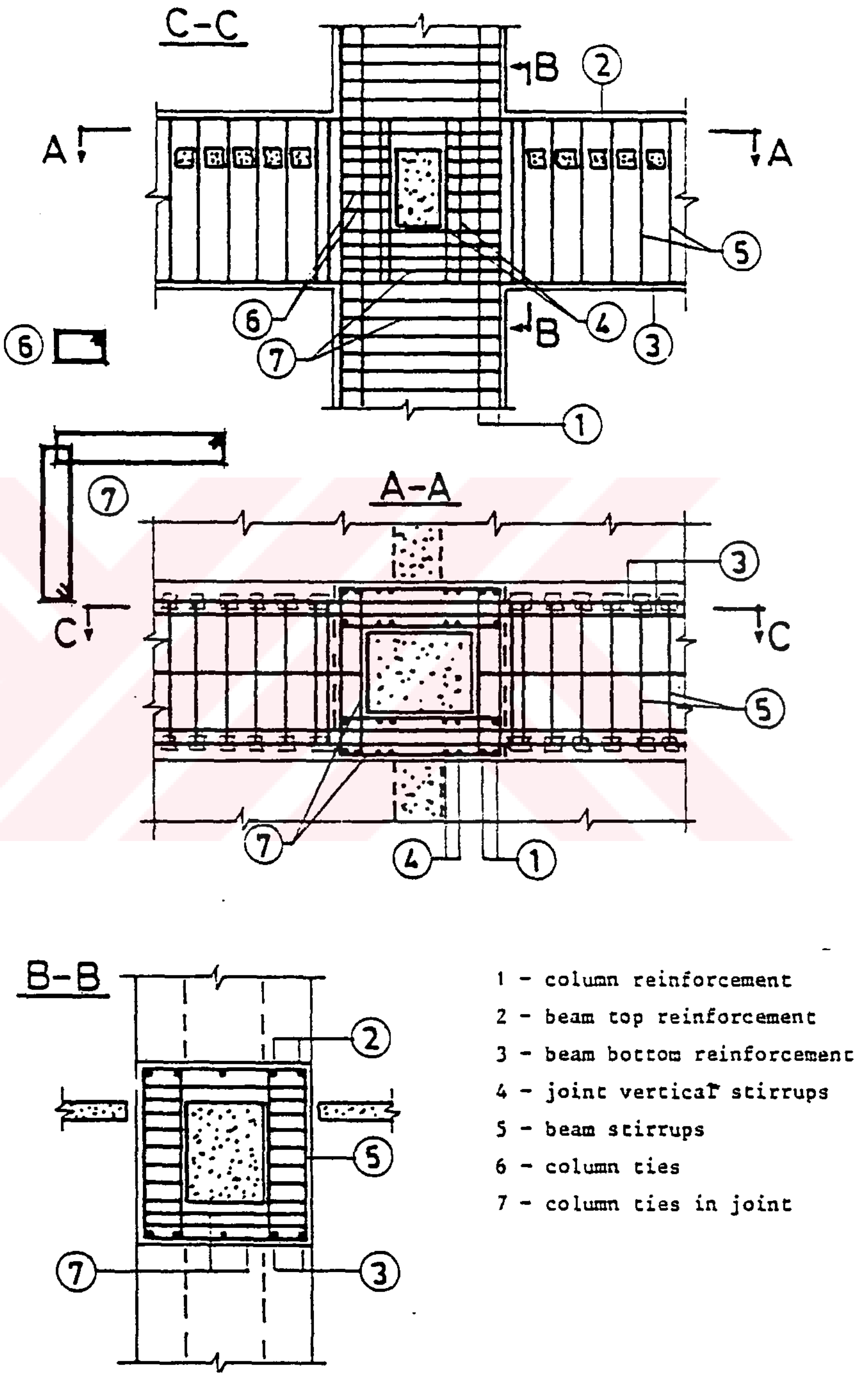
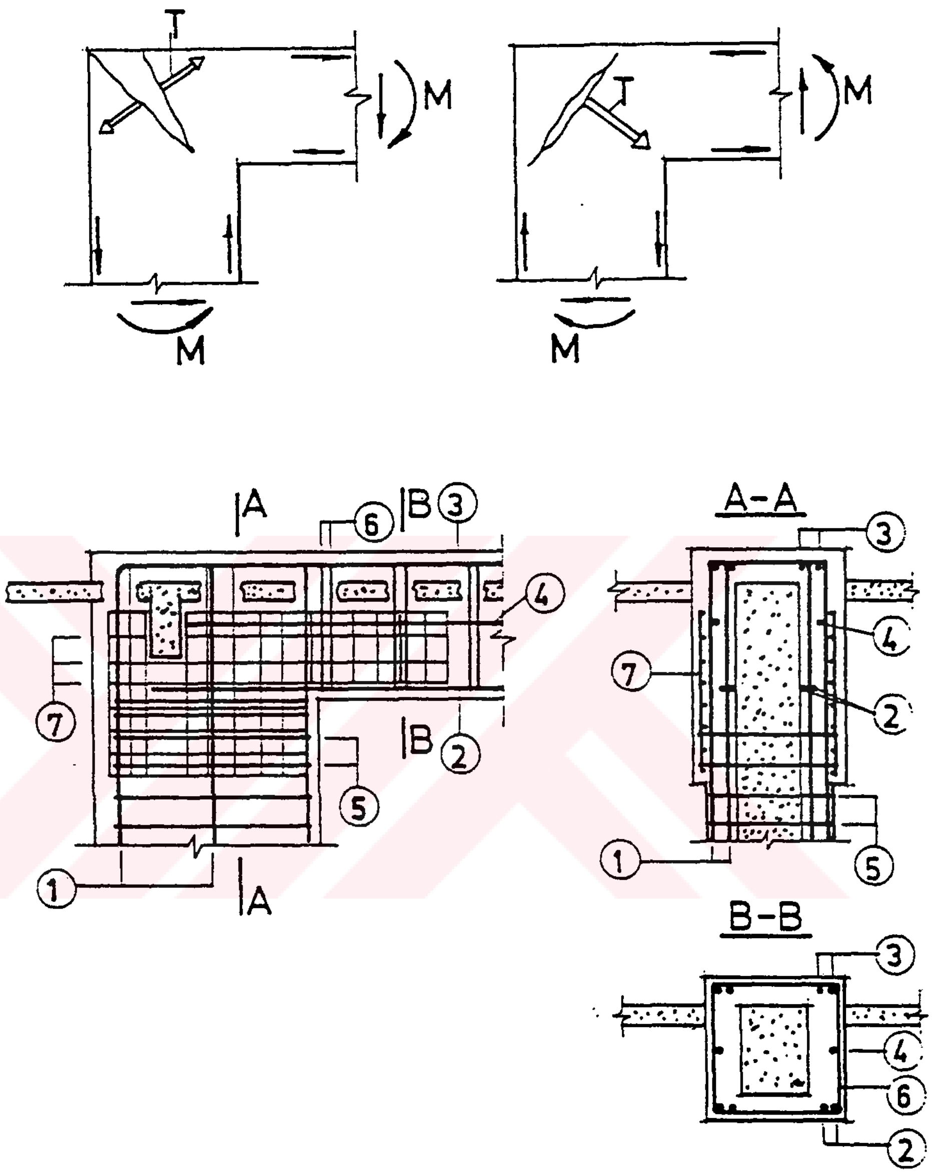


Fig. 7.14 Technique for strengthening heavily damaged beam-column joint with concrete jacket and additional reinforcements and ties

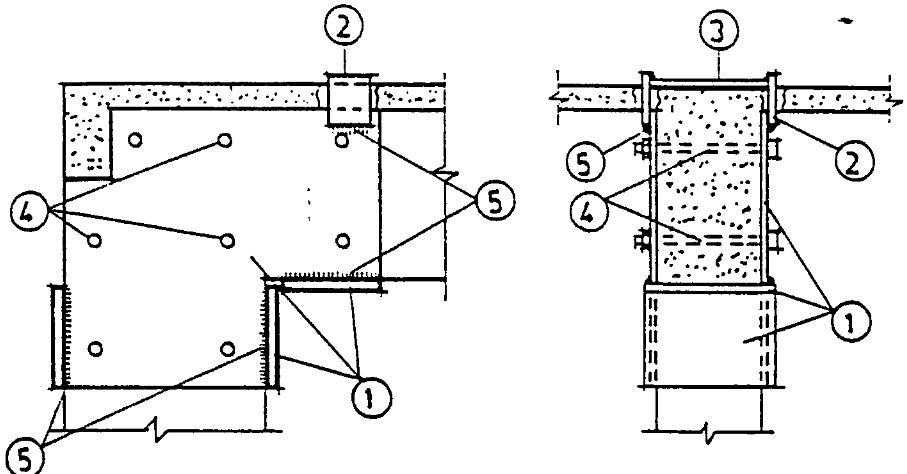


1 - column reinforcement; 2 - beam bottom reinforcement; 3 - beam top reinforcement; 4 - beam reinforcement; 5 - column ties; 6 - beam stirrups; 7 - welded wire fabric; T - tensile resultant

Fig. 7.15 Technique used for strengthening exterior column-beam joint by reinforced concrete jacket

7.4.2.2 Steel Plate Reinforcement:

Steel plate reinforcement is a technique which permits strengthening of joints without considerable changes in size. It can be used primarily in industrial type structures where heavy frames are located in one direction. Steel plates shaped according to the joint configuration are glued by epoxy resin to the concrete as shown in Fig. 7.16. The steel plates should be anchored to the joint by prestressed bolts. At uneven surfaces of the joint, a thin layer of expansive cement mortar for smoothing is advisable. Steel plate thickness must be at least 4 mm. Note that this technique is not practical when sizable beams frame into all four sides of a beam-column joint. The method will not provide reliable strengthening of the joint region for seismic resistance frames unless the steel plates are interconnected with similar steel plates or profiles used in strengthening the adjacent column and beam. External steel plate reinforcement should mainly be applied for strengthening procedures. Special measures must be taken into account for fire and corrosion protections.



1 - steel plate; 2 - steel plate; 3 - steel strap;
4 - prestressed bolts; 5 - welding

Fig. 7.16 Steel plate strengthening for column joint

7.5 Slabs:

Slabs, as we know they carry the vertical gravity loads and transfe them to the beams or columns and provide diaphragm action to resist the lateral loads by compatibility with all lateral resistant elements of the structure. Therefore, slabs must possess the necessary strength and stiffness. Damages in slabs generally occur in locations with irregularities, such as near large openings at concentrations of earthquake forces in slabs close to widely spaced shear walls and at staircase flight landings. Repair of slabs is necessary when damages occur. Strengthening is applied when there is insufficient slab strength or for increase strength in the region of newly-introduced shear walls.

7.5.1 Local Repairs of Slabs:

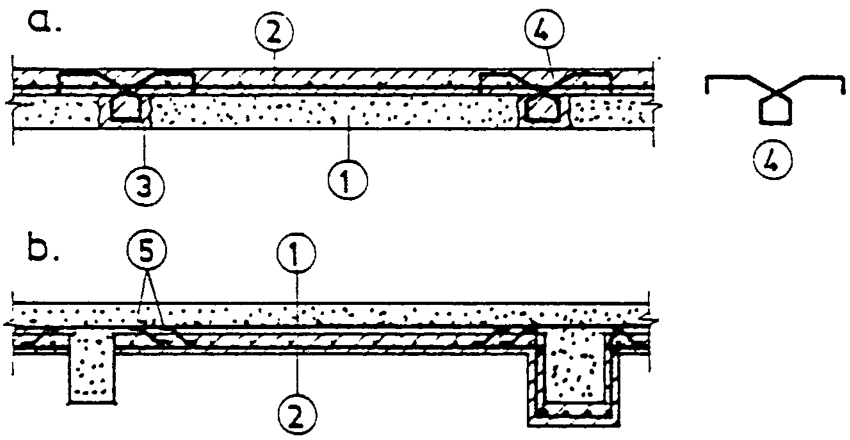
Repairing of slabs cracks can be achieved by the same methods in repairing columns and beams. In cases of spalled concrete, broken or buckled reinforcement and corrosion of reinforcement can be repaired by removal of all deteriorated materials and replaced by welding splice. Concrete with the same properties as the existing concrete should be used.

7.5.2 Technique for Strengthening Slabs:

Strengthening by thickening of slabs should be applied in case of insufficient strength or stiffness. The new materials are incorporated above or under the existing slab. As shown in Fig. 7.17.a thickening done above the existing slab, the flexural strength is increased because of the increased effective depth and ability to add negative reinforcement as supports. In the second case as shown in Fig 7.17.b thickening under the existing slab, the flexural strength increase because of the newly added tension reinforcement. In the case of 'a' normal concrete is used; in the case of 'b' the application of shotcrete is more suitable. The strengthening according to case 'a' provides greater floor slab stiffness necessary for the horizontal diaphragm action and is strongly recommended as the preferred method of the two.

For the compatibility of the existing slab and the newly-added reinforced concrete, obtaining excellent shear bond is of great importance. It can be achieved by means of the following:

(a) Reinforced concrete lugs as shown in Fig. 7.18.a.
(b) Rough surface, realized by gluing of sand grains with epoxy resin as show in Fig. 7.18.a.
(c) Epoxied steel dowel bars as show in Fig. 7.18.b.
(d) Additional shaped concrete lugs in slab-voids as shown in Fig. 7.18.c.
(e) Steel dowels performed either by steel angle profiles anchored by power concrete nails or by epoxied bolts or wedge anchor bolts as shown in Fig. 7.18.d.



1 - existing slab

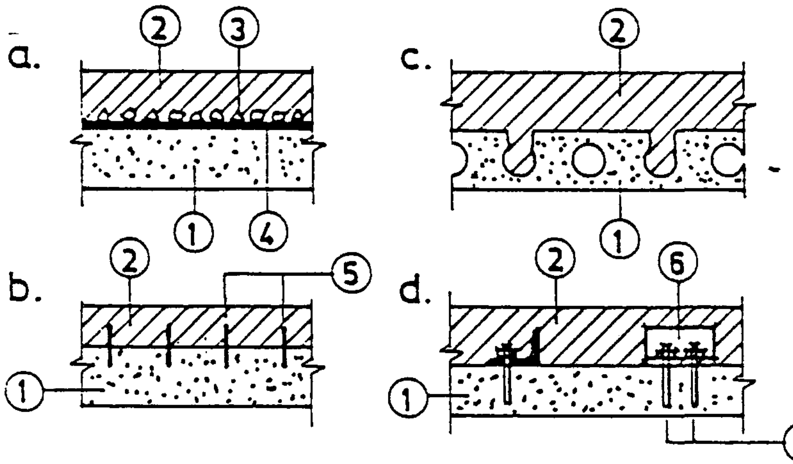
2 - added reinforcement

3 - dowel

4 - anchoring bent bars

5 - welded connecting bars

Fig. 7.17 Construction technique for strengthening slabs



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

Fig. 7.18 Technique for compatibility of the existing slab and the newly-added reinforced concrete

7.6 Shear Walls:

Shear walls are one of the most efficient element in a building in resisting lateral loads originating from wind or earthquakes, when they are situated in advantageous position in a building. Therefore, a severely damaged or a poorly designed shear wall must be repaired or strengthened in order that the structure's strength for seismic force can be improved. There are several experimental back data for techniques used in repairing and strengthening damaged and/or undamaged shear walls, which have been used effectively in Europe, and Japan.

7.6.1 Local Repairs of Shear Walls:

The purposes of repairing for a damaged shear wall are to make the strength and stiffness recover equivalent to the original vales. The following techniques are available to satisfy the above demand:

i) Injecting epoxy resin: into cracks of concrete in shear walls has become a standard practice over the last decade. When bond deterioration and concrete crushing have not occurred, epoxy injections are capable of restoring approximately the original wall flexural and shear strength. But this technique will never achieve the stiffness of the original wall. This is due to the fact that not all of the small cracks can be epoxy injected. Epoxy injections should be performed according to the method in repairing beams and columns, also the same recommendations in section 6.3 must be followed.

ii) Recasting concrete: in site after removal of a damaged parts from the damaged shear wall in case of large cracks. Corley W.G. et al. (1983) [15] studied the behavior of this technique and observed the following conclusions:

(a) Replacement of damaged concrete in webs of structural walls found to be a simple and effective repair procedure. Strength and deformation capacities of repaired walls were equivalent to those of original walls.

(b) Initial stiffnesses of repaired walls were approximately 50 percent those of original walls. Reduction in initial stiffness should be considered in evaluation of dynamic response of repaired structures.

The criterion used was to remove all concrete that could be taken out easily with hand tools. Repair procedures were chosen to provide the simplest and least expensive repair that would restore reasonable strength and deformation capacity to the wall. Also if there are buckled reinforcement must be repaired. The choice of replacement materials (polymer mortar, cement mortar, concrete or shotcrete) depends on the degree of damage, the desired repair characteristics and the rehabilitation conditions.

7.6.2 Techniques For strengthening Shear Walls:

Strengthening of existing shear wall should be applied when the original strength of damaged or undamaged wall is insufficient. There are different ways to add strength to an existing concrete shear wall. This technique can be achieved by shotcrete concrete or cast-in-place concrete with additional reinforcement. The following techniques are available to improve the strength of a shear wall:

i) Reinforced concrete jackets on damaged walls: used to repair and to strengthen a damaged concrete shear wall. Minor cracks must be repaired before adding the new jacket. Transversal shear connectors can be achieved by epoxying dowels in holes drilled in the wall. Generally, the recommendations for reinforced concrete column (in regard with surface preparations) are also valid for reinforced concrete walls. Minimum thickness to be used in case of shotcrete for single jackets 70-100 mm and for double jackets 50 mm, and also in case of cast-in-place for single jacket 100-150 mm and for double jacket 70-100 mm. The entire of the reinforced concrete wall has to be jacketed-up and special attention has to be paid on anchoring all new reinforcements and on connecting the jacket with the existing horizontal diaphragms.

ii) Increase of wall size: with reinforced concrete should be applied when the original strength of a wall is insufficient. There are different ways to add strength to an existing concrete shear wall Fig. 7.19.

When shear strength is to be increased thickening the web with additional reinforced concrete (Fig. 7.19.a). Horizontal and vertical bars are added depending on the strength increase desired. The reinforcement and the concrete must be anchored to the existing wall by appropriate roughening of the original concrete surface and by special anchor bolts.

When flexural strength is to be increased, reinforced concrete flanges must be added to both ends of the wall As shown in Fig, 7.19.b. The new flange must be confined by closely spaced hoops and cross ties. Anchorage of the new flange concrete can be achieved by welding of special bent bar connectors to the new and the existing reinforcement.

If both flexural and shear strength must be increased, reinforced concrete must be added to both parts, the web and the cross section ends as shown in Fig 7.19.d. Solutions with one-sided (Fig. 7.19a) or tow-sided (Fig. 7.19.d) web thickening are possible.

As shown in Fig. 7.20 an example of a two-sided shear wall strengthening. Shear forces between the shear wall and the floor slab must be adequately transmitted. For this purpose, concrete dowel connections are made by opening holes through existing slab. These holes also serve for concreting the new wall parts beneath the slab. Diagonal reinforcement bars passing through the slab and anchored in the upper story and lower story walls provide an additional connection for shear transfer.

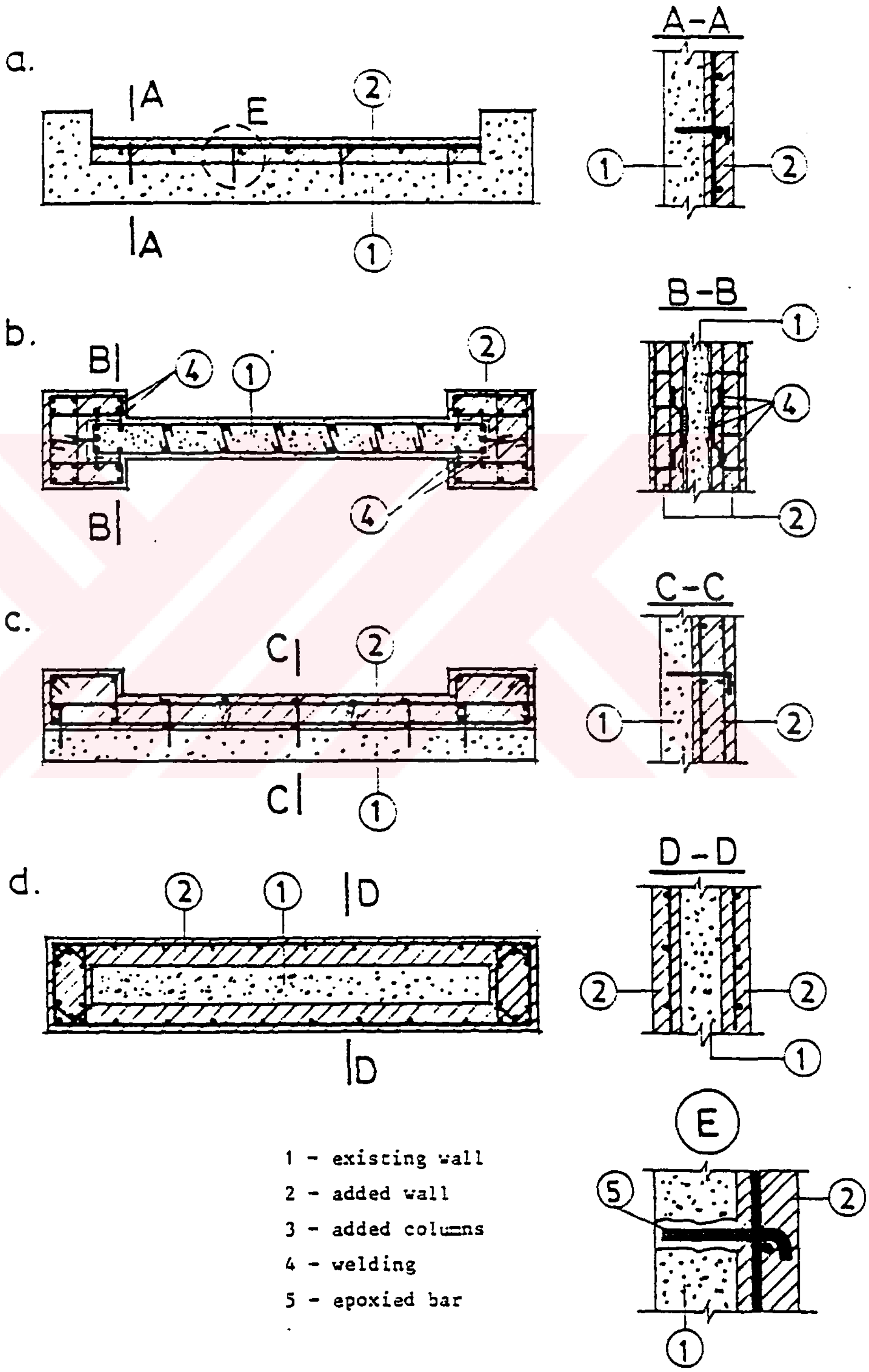
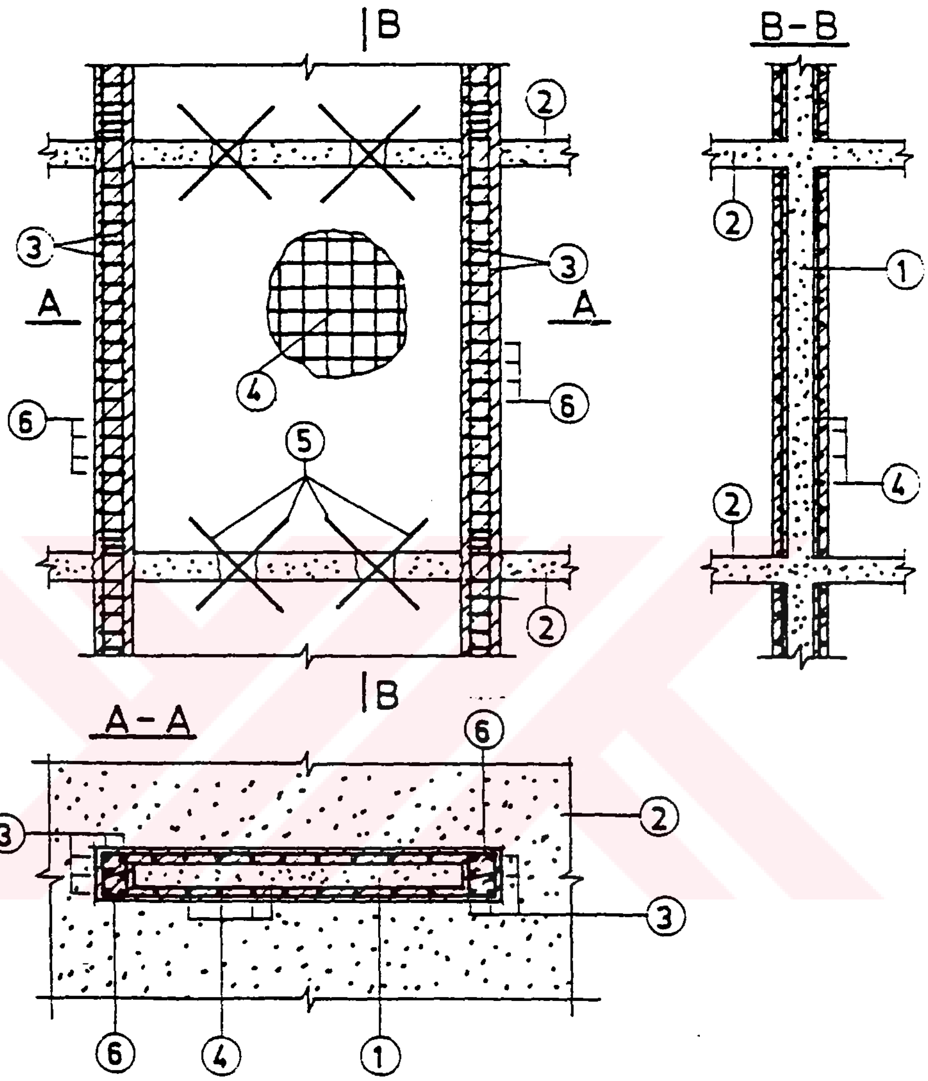


Fig. 7.19 Technique for strengthening existing shear wall by reinforced concrete jacket



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

Fig. 7.20 Two sided reinforced concrete jacket for strengthening existing shear wall

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

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

(a) The strength of the new materials should not be less than those of the existing walls.
(b) The web thickness of the new material should be at least 50 mm. The thickness of the new shear wall flanges should be at least 100 mm.
(c) Both the horizontal web reinforcement area and the vertical web reinforcement area must be no less than 0.0025 times the gross area of the wall thickening.
(d) The area of the vertical reinforcement concentrated at the wall ends must be no less than 0.0025 times the gross area of the newly added cross section.
(e) The wall end ties diameter should be no less than 8 mm or 1/3 of the vertical end wall reinforcement bar diameter. The tie spacing should be no more than the thickness of the wall end thickening or maximum 150 mm.
(f) The new concrete should be anchored to the eisting concrete with epoxied hooked dowels at a maximum of 600 mm in each direction and the existing wall surface should be thoroughly roughened.

7.7 Introduction Of New Structural Elements:

There are several experimental research studies, for introducing or adding a new structural elements to increase the lateral force capacity of existing structure. Most of them have a reliable results and have been used effectively in Europe, Japan, South and North America. These techniques have been used to increase capacity by adding new structural elements, to resist part or all of the seismic forces of the structure.

The newly added elements may be subdivided as follows:

1- Infilling new shear walls into thxe existing frames.
2- Adding side walls to the existing columns.
3- Infilling steel braces into the existing frames.
4- Adding buttresses to the exterior frames of a building.
5- Additional frames in a frame or skeleton structure.

The choice of the type, number and size of the added elements depends on the particularities of the existing structure and the functional layout of the building. Additional of new structural components in an existing building will change the dynamic behavior of the whole space structure considerably during an earthquake. The increase in stiffness will also tend to increase the seismic design forces for most typical structures. It also cause considerable redistribution of the lateral forces between the earthquake resisting elements. Therefore, it is very important that the most favorable conditions be created whenever possible, as follows:

(a) Avoiding large concentration of forces in members with small strength and/or ductility capacities by locating the strengthening elements uniformly throughout the structure.

(b) Improving the distribution of lateral force by reducing the effects of torsion and irregularities

(c) Providing sufficient strength, stiffness and ductility of the individual elements and of the whole structure.

(d) Providing adequate strength in connections between the existing structure and the newly added elements.

(e) Providing stiffness compatibility of the existing and the newly added elements.

Additional structural elements must be introduced in regular way in plan and elevation, also should be placed in such a manner as to minimize the distance from the center of mass to the center of rigidity. More attention must be paid to temperature, shrinkage and creep which can produce significant redistribution in the internal forces between the existing and the new structural elements.

7.7.1 Infilled Walls:

Introduction of various types of infills improves the seismic resistance of framed structures and causes an increase in the lateral stiffness leading to a reduction in interstory drift. There are several experimental back data for strengthening by infilling shear wall, such as cast-in-place reinforced concrete infills, masonry, precast panel wall and multiple precast panel wall. Structurally, the infilled wall is a shear wall consisting of the existing frame working together with the infill. But the most popular techniques of the mentioned above is the in-filling reinforced concrete shear walls by cast in place concrete into the existing frames.

7.7.1.1 In-Filling Shear Walls:

Strengthening of framed reinforced concrete structures by cast-in-place reinforced concrete infills is commonly used in practice. There are many experimental data for this technique. This technique is the most reliable one to increase the seismic capacity. The in-filling shear walls includes, filling existing frame by new reinforced concrete shear wall, filling opening of existing reinforced concrete wall with new concrete and to make a monolithic wall and to increase the original wall thickness with new concrete[14] as shown in Fig. 7.21

The new reinforced concrete shear walls are jointed to the existing reinforced concrete frame or wall by the post-instulled anchors or the concrete shear keys. Connections with the column can be detailed by the following methods:

(a) Chipped keys in the existing columns. Additional anchor bars are welded to the longitudinal column reinforcement (Fig 7.22.a).
(b) Chipped keys in the existing columns. Diagonal anchor bars are welded to the longitudinal column reinforcement and additional ties are welded to

the existing column ties (Fig. 7.22.b).

(c) Chipped keys in the existing column plus steel dowels epoxied in the existing column **Fig. 7.23.a** Additional longitudinal reinforcement can be added depending on conditions at the floor beams **Fig. 7.23.b.**

(d) Encasement or jacketing of existing column if the column must be strengthened for compression **Fig. 7.23.c.**

Connections with beams can be detailed by the following methods:

(a) Epoxied bolts in predrilled holes into the beams **Fig. 7.24.a.**

(b) Combination of chipped keys on the existing beam with additional anchor bars, welded to the existing reinforcement and epoxied bolts in predrilled holes **Fig. 7.24.b.**

(c) Combination of chipped keys on the existing beam with additional diagonal anchor bars welded to the existing longitudinal reinforcement and additional stirrups welded to the existing ones **Fig. 7.24.c.**

(d) Steel dowels constructed by welding together steel plates attached to the existing beam by epoxy glue and wedge anchor bolts **Fig. 7.24.d.**

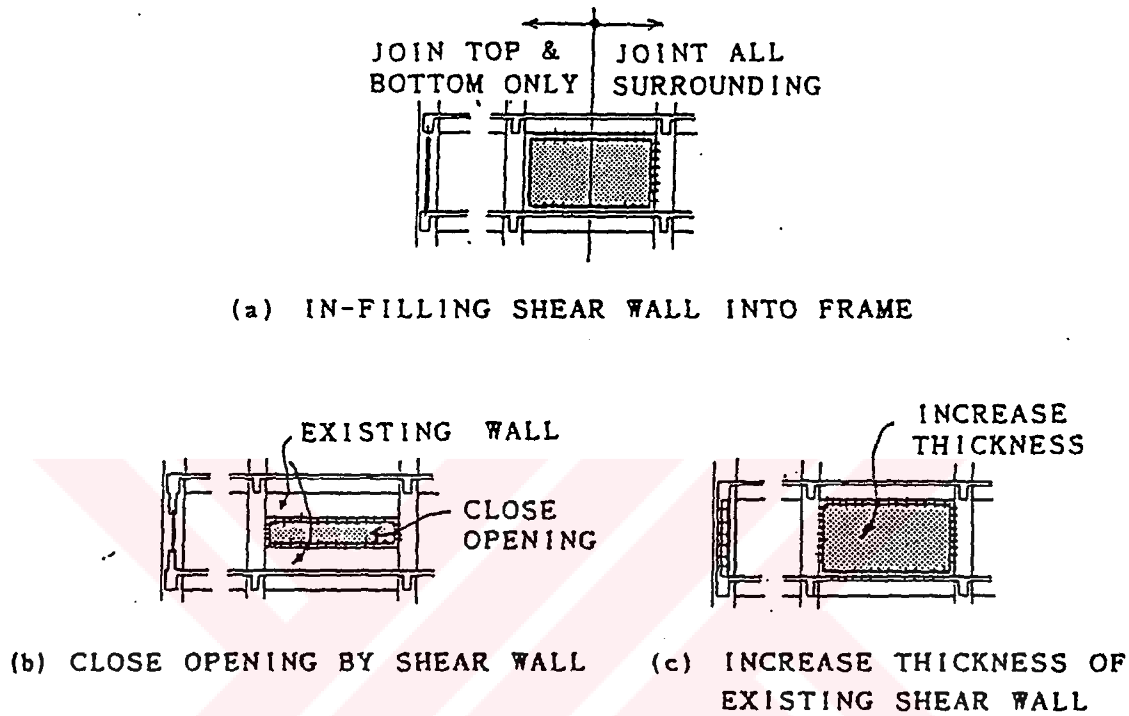


Fig. 7.21 Techniques by in-filling shear walls

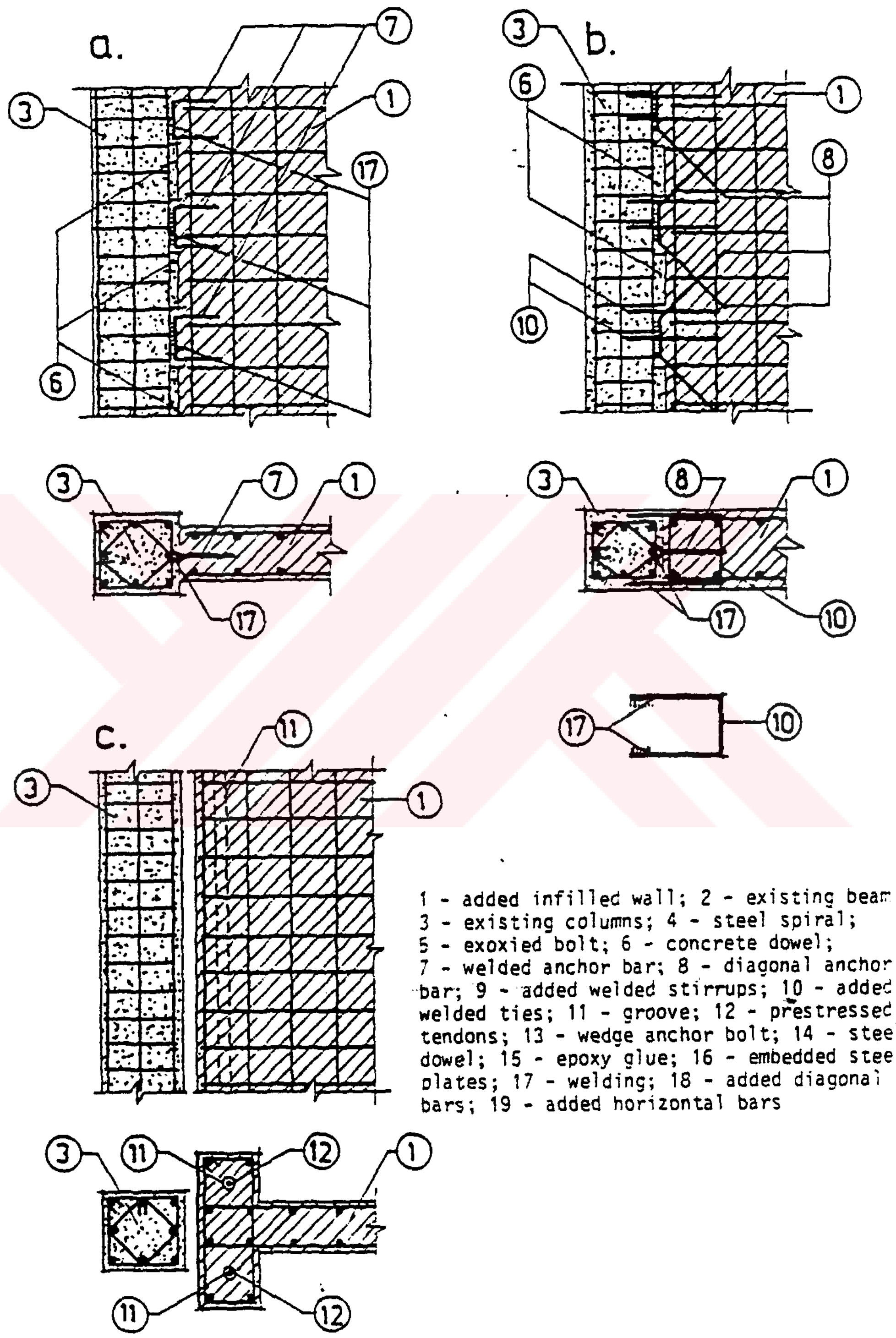


Fig. 7.22 Techniques for connection of in-filling shear wall to the existing columns

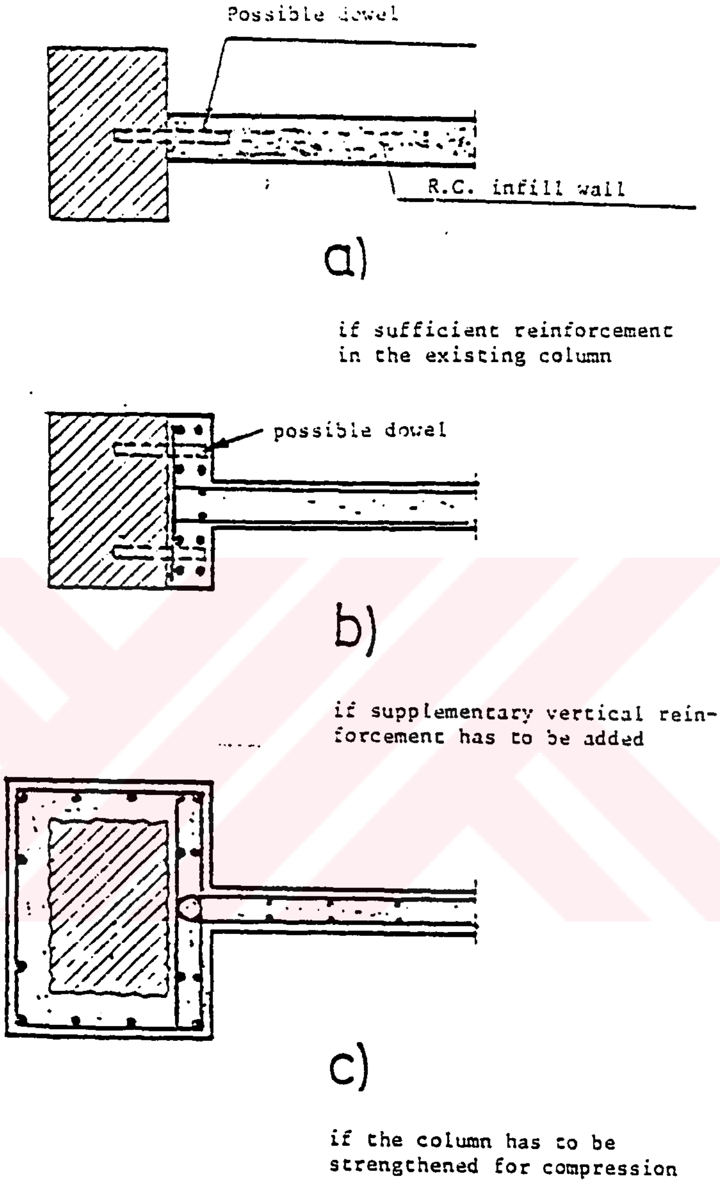
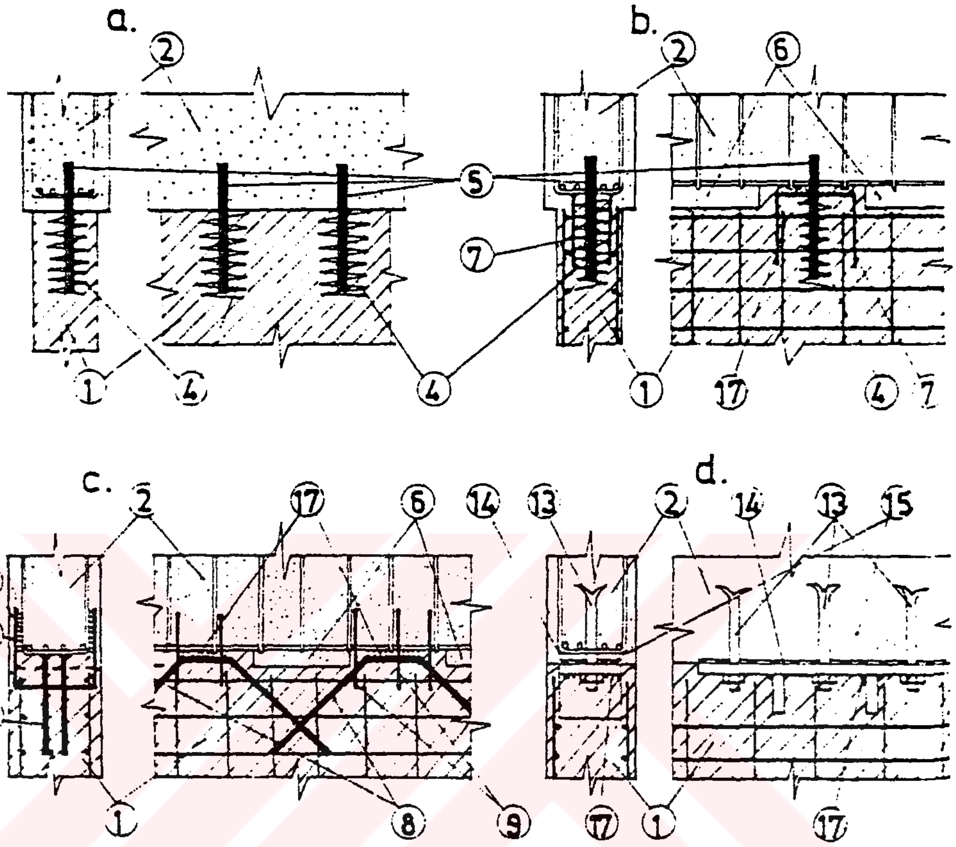


Fig. 7.23 Construction techniques for connction of new shear wall to the existing column



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

Fig. 7.24 Technique for connecting in-filling shear wall to the existing beams

7.7.2 Adding Side-Walls to The Existing Frames:

This technique is used to increase the lateral strength of existing columns by adding narrow walls to each side of an existing column [14] as shown in Fig. 7.25. The side wall generally has a thickness considerably less than the width of the adjacent column, and must be placed symmetrically about the existing column. Connections as previously discussed under jacketing and infilling are appropriate for side walls. However, the shear strength of boundary beams should be checked, because the clear span of existing boundary beams became shorter than before and the failure mechanism might change after strengthening. According to the Japanese recommendations [14] "Guidelines For Retrofit Techniques Of Existing Reinforced Concrete Building" the following details must be considered and followed such as :

(a) Side-walls should be added symmetrically to both sides of a column, in general.
(b) The minimum width of a side-wall should be wider than the half of original column depth, and wider than 500 mm, but should be wider than the double of original column width.
(c) The minimum thickness of a side-wall should be more than 1/3 of original column width, and more than 200 mm, but should be less than the boundary beam width.
(d) The aspect ratio of a precast side-wall, L/H_o, should be more than 1/3, and the width should be more than 800 mm.
(e) The thickness of a precast side-wall should be more than 150 mm.
(f) The reinforcement ratios in a side-wall should be

more than 0.25% in both the horizontal and vertical direction.

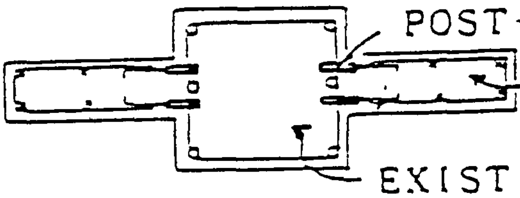


Fig. 7.25 Seismic retrofitting with added side-walls

7.7.3 Infilling Steel Braces Into Existing Frames:

The techniques by adding steel braces or steel panels into the reinforced concrete frame have often been applied in this decade. [14] This technique are used when there are need to keep the lighting through windows like the case in school buildings. This technique is also effective to increase the seismic capacity of a building, and there are many experimental data to refer the details. [2,3,14] If the strength capacity of the existing column is insufficient, which is usually the case, a full vertical truss with vertical chords can be added. If column and beams have sufficient strength and ductility, the steel diagonal braces can be added within the concrete frame to form a vertical truss of existing beams and columns and new diagonals.

There are some variations or combinations to apply the steel bracing systems into the existing reinforced concrete frames as the following: [14]

1- The technique using the special steel frame to set the steel braces or panels into the existing reinforced concrete frame.

2- The technique not using such the special steel frame.

3- The technique using the steel bolts or welding to frame, called as the DIRECT JOINT TECHNIQUE.

4- The technique using joint mortar or concrete and the post-installed anchors to fix the steel braces, called as the INDIRECT JOINT TECHNIQUE.

Strengthening of existing building by the steel bracing system is basically achieved by setting steel braces or panels into the existing reinforced concrete frame as shown in Fig. 7.26. The installation of steel anchors at the inside of existing reinforced concrete frame is required first. If there are some existing non-structural walls, they should be removed before the installation of anchors. Then the steel bracing system, that was fabricated in a factory in advance and has the steel stud bolts at the periphery of steel frame, is set into the existing reinforced concrete frame in the site. Finally the joint mortar or concrete is injected into the joint between the existing frame and the steel bracing system. The steel bracing system usually should be covered by another finishing materials for the rust proof and fire resistance.

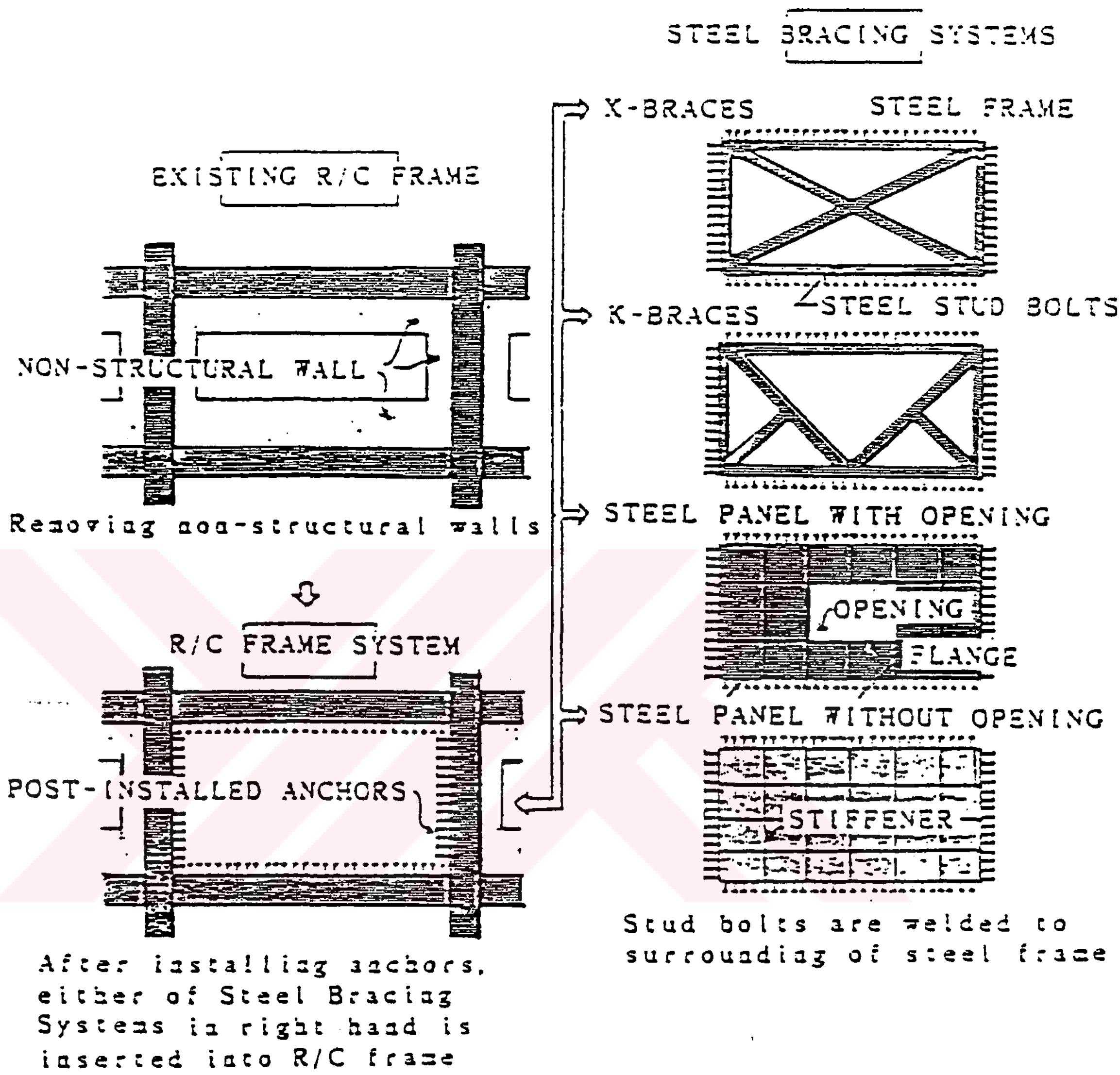


Fig. 7.26 Techniques for strengthening by steel braces

One of the details for joint between the existing frame and the setting steel frame is shown in Fig. 7.27 The technique shown in this figure is called the INDIRECT JOINT TECHNIQUE, where the force from the existing frame to the steel frame is transferred indirectly through the compression or shear stress in the joint concrete. [14]

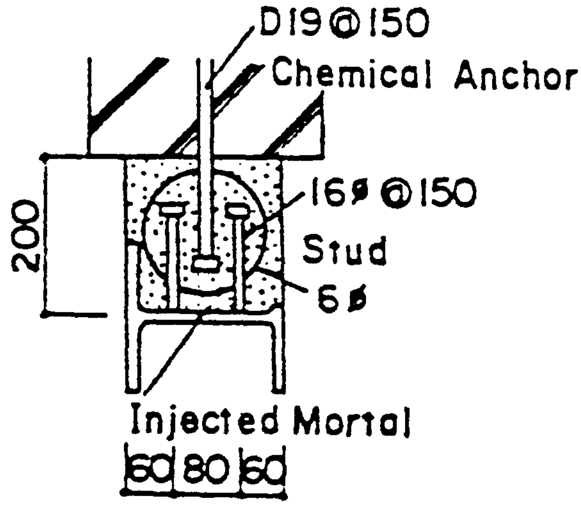


Fig. 7.27 Technique for post-installed anchor "indirect joint technique"

The following details should be required for strengthening by steel bracing system:

(a) The K-type or X-type should be selected for the shape of steel braces, and the slenderness ratio of a bracing member should be less than 58.

(b) The gauge line between the steel braces and the surrounding frame should be coincided.

(c) The details for the post-installed anchors should be followed by the coming chapter.

(d) The post-installed anchors generally should be provided at the inside of whole surrounding existing frame, and the surface of existing reinforced concrete frame should be roughened by chipping the concrete.

(e) The joint concrete or mortar between the existing reinforced concrete frame and the steel bracing system should be confined by the spiral wires or welded wire fabrics.

(f) The reinforcement ratio of spiral wire or wire fabrics in the joint concrete, p_w should be equal to or more than 0.4% using $p_w = A_w/h.x_w$, (A_w: area of wire, h : clearance between steel frame and concrete surface, x_w: pitch of wire).

(g) The compressive strength of joint concrete or mortar should be equal to or more than 30 MPa.

(h) The roll iron whose size is equal to or larger than H-150x150x7x10 should be used as a brace member.

(i) The thickness of steel plate for a panel should be equal to or more than 4.5 mm, and the panel should be strengthened by the steel plate stiffener that must be arranged with the 1000 mm or less pitches.

(j) The diameter of post-installed anchor should be equal to or more than 26 mm, and the pitch should be equal to or less than 250 mm.

(k) The diameter of steel stud bolt with head should be equal to 16 mm or 19 mm, and the pitch should be equal to or less than 250 mm.

7.7.4 Adding Buttresses to The Exterior Of Frame Building:

This technique is achieved by adding buttresses to the existing building from outside as extension of the frames. Adding buttresses are not so popular because they may require the vacant site around the building, and the enough reaction from the piles or foundation of the buttress should be required. As shown in **Fig. 7.28** the buttress is achieved by reinforced concrete with enough anchors to the existing frames. This technique applied to increase the lateral force capacity of a building. According to Japanese recommendation "mentioned Above the followings are required to strengthen effectively.

(a) The buttresses should be extended symmetrically to both ends of a existing frame.

(b) The columns or beams should be provided to the boundary of a buttress to confine the wall in a buttress. The existing column should be connected strongly with buttress.

(c) The buttresses in each side of a frame should be connected together by slabs to make sure the shear transfer by the slabs.

(d) The buttress should have a new foundation. The foundation should be supported by the hard layer in the ground or strong piles not to make any settlement. The foundation should be connected to the existing foundation together by a footing beam.

(e) The joint between the buttress and the existing frame should support the necessary tension and shear force expected.

(f) The thickness of wall in a buttress should be greater than 150 mm. The shear reinforcement ratio in a wall should be more than 0.2 percent.

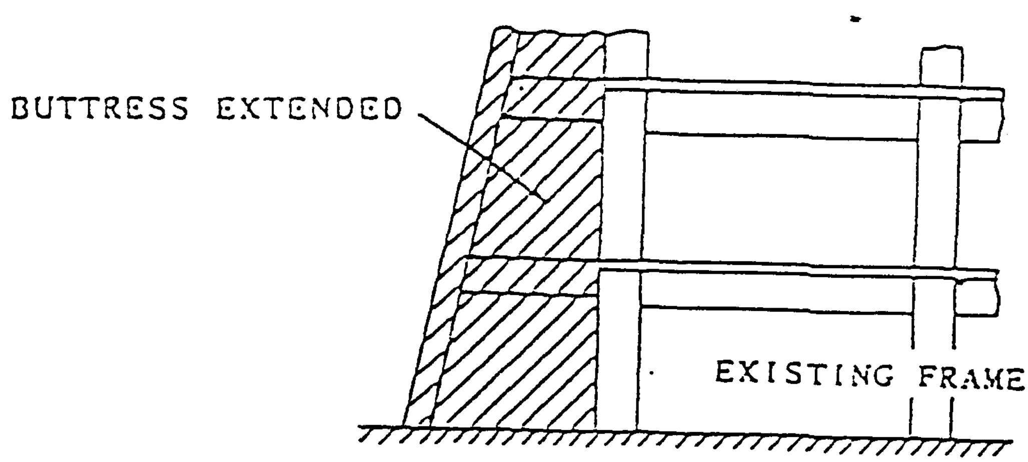


Fig. 7.28 Seismic retrofitting with added buttresses

7.8 Foundations:

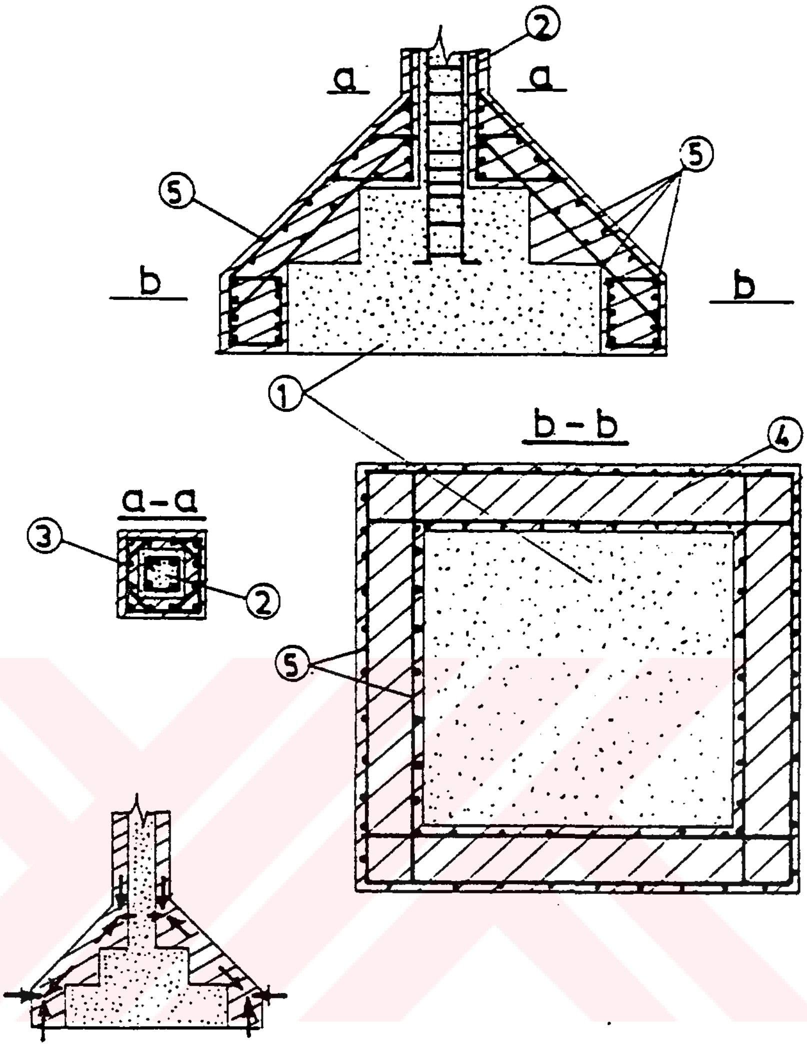
Repair and strengthening of foundations are usually not easy, and also they need expensive construction procedure. It should be performed in the following cases: [1]

1- Excessive settlement of the foundations due to poor soil conditions.
2- Damage in the foundation structure caused by seismic overloading.
3- Increasing of the dead load as a result of the strengthening operations.
4- Increasing the seismic loading due to change in code provisions or the strengthening operations.
5- Necessity of additional foundation structure for added floors.

The technique of repair and strengthening of foundations, can be performed by the following:

(a) Strengthening existing foundations.
(b) Adding new foundations.
(c) Modifying the soil for improved foundations support.

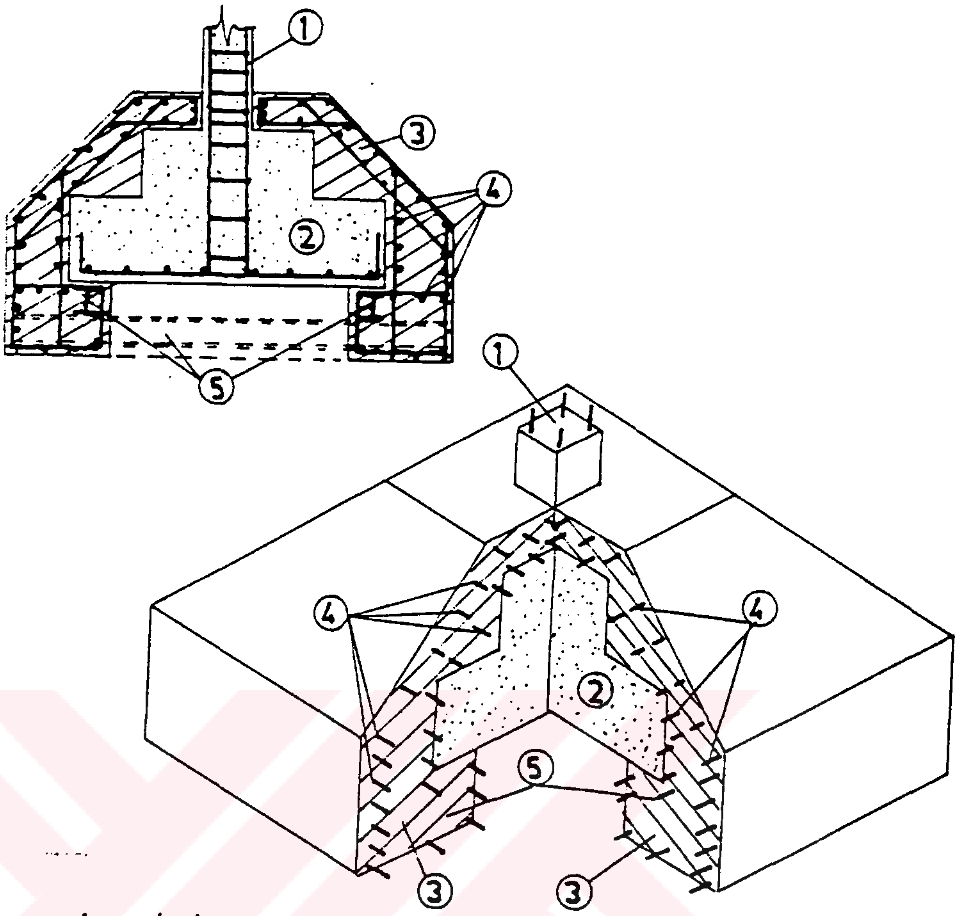
When there is a necessity to increase the footing dimensions due to increasing the bearing area of footing, the additional soil pressure must be equalized in the footing body. In case of columns strengthening with jacketing as shown in Fig. 7.29. The footing belt, arranged at the bottom, is of great importance for transmitting the inclined forces (in strengthening upper part) to the soil. Therefore, strong belt reinforcement with adequate splicing is necessary.



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

Fig. 7.29 Technique for strengthening foundations

In the case of an unstrengthened column, soil pressure applied to the strengthened part must be transmitted directly to the existing foundation body as shown in **Fig. 7.30**. This can be achieved by incorporating a steel profile under the end of the existing footing. Adequate reinforcement must be placed in the newly added footing body. [1]

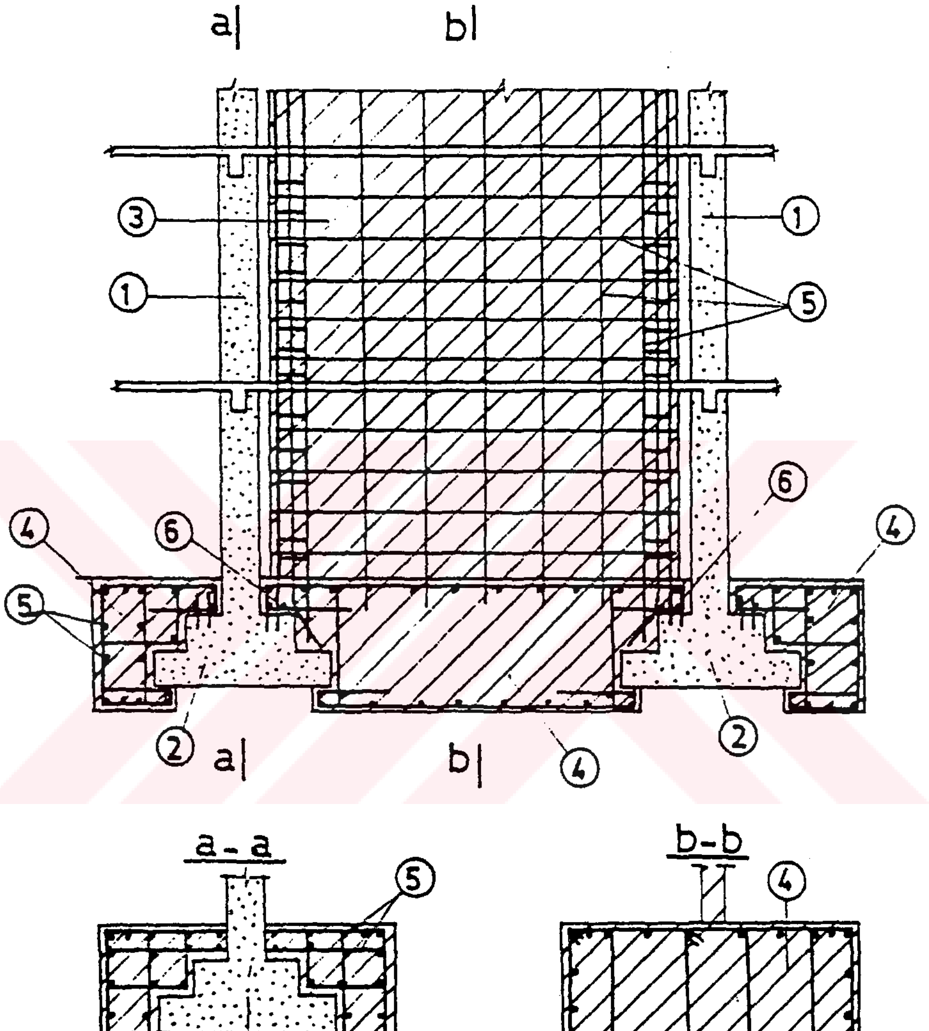


1 - existing column; 2 - existing foundation; 3 - added concrete;
4 - added reinforcement; 5 - steel profile

Fig. 7.30 Technique for stremgthening foundations in case of an unstrengthened column

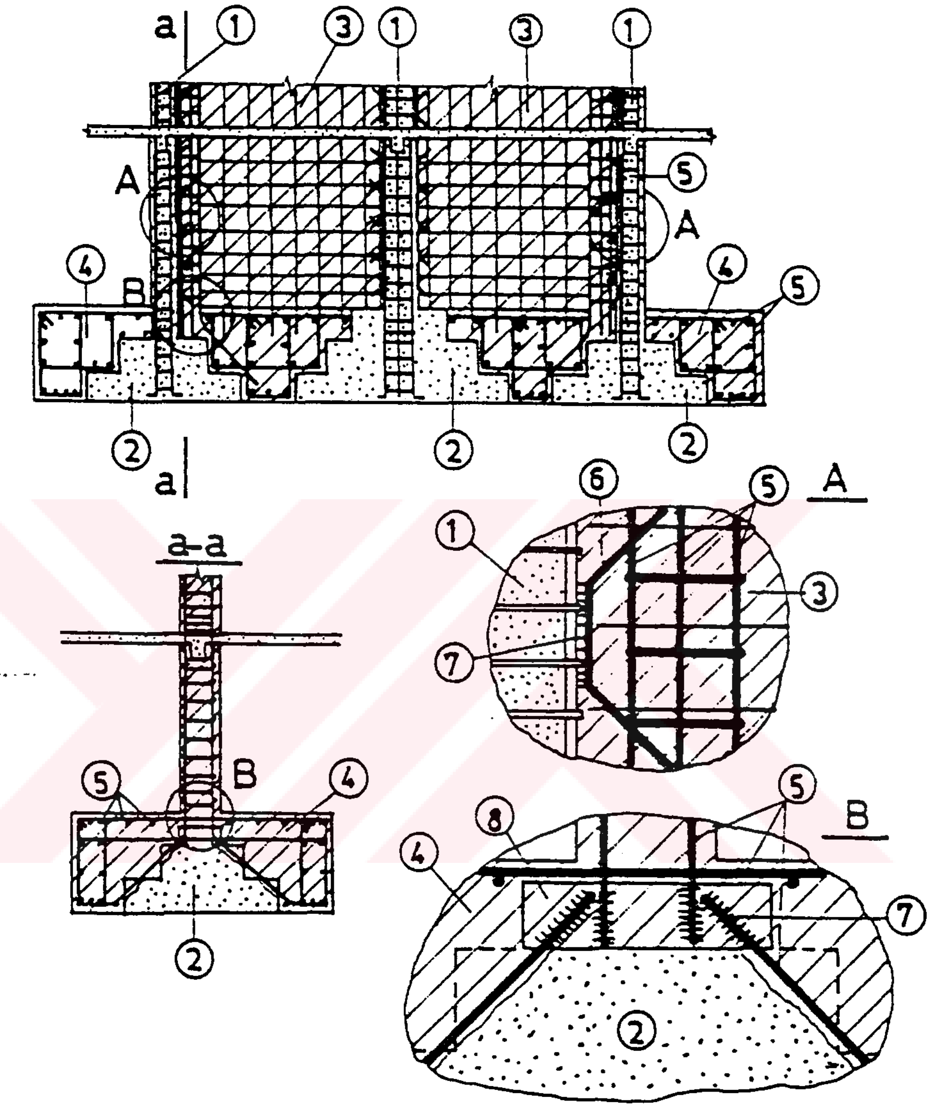
Adding new foundations is necessary when shear walls or similar elements are constructed in the building structure as show in Fig. 7.31 and Fig. 7.32. Special attention should be paid to incorporating the existing footing into the newly introduced foundation structure in such a manner that the parts will function together properly. The anchorage of the main vertical shear wall end reinforcement into the foundations is very important. This can be accomplished either by anchorage of the reinforcement in predrilled holes in the existing foundations with epoxy resin or by inclined reinforcement bars in the newly constructed foundation parts. The inclination of the reinforcement bars can be accomplished by welding to steel plates (Fig. 7.32.b) with adequate ties to resist any unbalance in lateral components of force. The axial forces from the overturing moment must be resisted by adequately detailed reinforcement placed in the newly constructed foundations. The wall shear forces must be transmitted to the foundations by sufficiently anchored web reinforcement.

Modifying the soil for improved foundation support can be performed by compaction grouting in case of fine grained soils, or by chemical solidification in case of relative permeable granular material. compaction grouting results in densification of the soil and, therefore, it reduces the potential for liquefaction. Providing cohesion in granular soils by chemical solidification also reduced the potential for liquefaction.



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

Fig 7.31 Construction technique for sttengthening foundation with in-infilling shear wall



1 - existing columns; 2 - existing foundations; 3 - added cast-in-place infilled shear wall; 4 - added concrete; 5 - added reinforcement; 6 - diagonal anchor bars; 7 - welding; 8 - steel plate

Fig. 7.32 Technique for anchorage of new shear wall in the existing foundations

Chapter 8

DESIGN PROCEDURES FOR REPAIR AND STRENGTHENING OF R.C. BUILDING

8.1 General:

Rational design of existing damaged and/or weak structure is unanimously acknowledged as one of the most difficult problems that the engineer has to face, and there is certain number of reasons for it: uncertainties related to the assessment of existing structures, as well as the unconventional analytical models needed, also his knowledge of the structure is generally very incomplete. [2] The safety level of a damage structure and also the accepted ways to demonstrate this level are not settled by any technical regulations. Experimental as well as theoretical studies have been carried out, in order to establish design method for upgrading. Temporary provisional conclusions and practical guide rules are given for redesign purposes, with emphasis on the real characteristics of the final sections after interventions as compared to the corresponding ones of equal but monolithic sections. When a building is to strengthened or repaired, its projected new response characteristics must be carefully analyzed so that the selected procedure does not create new weaknesses. The strengthening and repairing must also be cost-effective. Information on the relative merits of different strengthening and repair procedures is needed for the design process of repair and strengthening. In general , while working in redesign and especially when dimensioning additional structural arrangements, it should be noted that the following two aspects have also to be considered: [2]

a) Serviceability requirements should be also be taken into account, i.e. criteria concerning the adequacy of final stiffnesses and criteria concerning the width of cracks of repaired and/or strengthened building elements have to be sought.

b) Durability should also be thoroughly considered, i.e. properties of existing as well as of additional materials under aging-environmental actions and imposed deformations (especially thermal-effects and additional problems of time-effects due to the intervention itself).

The following basic steps need to be taken in redesign of repair/strengthening:

1- Chose criteria of repair and strengthening.

2- Damage evaluation and selection of repair and/or strengthening solutions.

3- Final design procedures.

8.2 Criteria of Repair and Strengthening:

The design criteria for the development of concepts for upgrading of existing building, especially seismic upgrading of existing building will be in accordance with the applicable provisions as required for new construction. Each country or governmental region should establish its own criteria based on specific conditions related to the circumstances of the region.

8.3 Damage Evaluation and Selection of Repair and Strengthening Solution:

Structural engineers in charge of damage classification and evaluation, they can use the methods mentioned in chapter 2, and chapter 5 or similar techniques. The Japanese technique [16,17] is one of the best techniques used for interventions after an earthquake to classify the damage degree and judgment for necessity of strengthening, for medium and low-rise reinforced concrete buildings.

After the investigation has been documented and the criteria for repair and strengthening, the designer must typically evaluate the damage, perform analysis to determine, why the damage occurred, and develop alternate schemes to repair and/or strengthen the structure. These alternative schemes must be evaluated and the most appropriate solution selected.

In general in most of the cases of damage evaluation, the engineer involve in designing of repair and strengthening, he must first analyze the damaged structure and thoroughly understand why the damage occurred. The force resistant paths in the building must be determined and it must be explained why certain member sustained damage while other member were essentially undamaged. It must be determined if the structure suffered due to discontinuities in strength or stiffness, due to torsional moments within the structure, due to hammering with adjacent structures or due to improper connections or details. The effect of non-structural elements such as infilled walls and appendages on the structural performance must be considered. It must be determined if members failed due to shear, compression, tension, flexure, bar anchorage, etc. This analysis is essential before any repairs can be assigned. [1]

Based on the calculation and analysis solution of repair and/or strengthening can be selected, a complete analysis with the appropriate mathematical model, according to the defined criteria, must be performed for the design. According to the experience gained from past earthquake, more attention should be paid to the weak links in concrete building such as: [1]

1- Short, stiff columns in comparison to other columns at the same level, possibly due to the presence of partial height infilled walls.
2- Columns beneath discontinuous shear walls.
3- Abrupt changes in stiffness due to changes in layout or structural system.
4- Concrete frame joints at infilled walls.
5- Stories which are more flexible than adjacent stories, possibly due to less infilled walls.
6- Excessive building torsion due to inappropriate distribution of stiffness.
7- Joint regions in concrete frames.
8- Flat slab to column connections.
9- Precast elements with weak connections.

8.3.1 Evaluation of Seismic Capacity of Existing R.C. Building:

As a consequence of the earthquakes over the active zone in the world, most of the countries which are found in active seismic zone, they developed their own methodology for the seisic evaluation of existing building, with the purpose of detecting those with the most vulnerability, in order to make the preventative actions necessary to improve the security of the citizens in future earthquakes. The available methodology such as:

1- Building Construction under Seismic Conditions in the Balkan Region, "Vol. 4 Post-Earthquake Damage Evaluation and Strength Assessment of Building under Seismic Conditions, 1985". [3]
2- Japan Building Disaster Prevention Association, "Guideline for Evaluation of Seismic Capacity of Existing Reinforced Concrete Building, 1977,revised in 1990". [17]
3- The applied Technology Council, "ATC-14: A methodology for the Seismic Evaluation of Existing Buildings, California, USA, 1971". [18]

These methodologies mainly depend on the base shear strength of the main structural elements, the connection strength and ductility of the connections, the building configuration, the material type, and the interconnection of the structural parts.

The Japanese develop a practical method to evaluate the seismic performance of existing medium low-rise reinforced concrete building in sort of indices. The basic concept of the method [17] as follows:

<u>**Unified seismic performance index of structures:**</u> the unified seismic performance index of structure (I_s) up

to six stories is evaluated by the following equation at each story and to every frame direction of a building:

$$I_S = E_o \cdot G \cdot S_D \cdot T \qquad \text{..............(8.1)}$$

where, E_o= basic structural index calculated by ultimate horizontal strength, ductility, number of story and story level considered;

G = local geological index to modify the E_o-index;

S_D= structural design index to modify the E_o-index due to the grade of irregularity of the building shape and the distribution of stiffness;

T = time index to modify the E_o-index due to the grade of the deterioration of strength and ductility.

The overall method consists of three level procedures; first, second and third level procedures. The first level procedure is the simplest, but most conservative of the three, while the basic concept is common for all three.

Note: for more detail found in reference [17].

8.4 Final Design Procedures:

Final design procedures include the detailed calculations of the styrengthening solution and the preparation of drawings, specifications and instructions so the work can be accomplished. [1]

The completed design for repair and strengthening should be clearly presented in complete construction drawings, instructions and details. The designer should keep in mined that repair techniques may be unfamiliar to some construction workers and detailed procedures and instructions must be provided. The documents must show all work that is to be performed without leaving interpretation to the contractor and the workmen. If existing conditions are unknown, the drawing can require exposure of the unknown situation and notification of the designer for determination of the repair details. Schedules listing all columns, beams and other elements in the structure with reference to particular repair or strengthening details or procedures can be helpful in clearly defining the full scope of work. [1]

Chapter 9

CALCULATION TECHNIQUES AND DIMENSIONING OF ADDITIONAL STRUCTURAL ARRANGEMENTS

9.1 Columns:

In this strengthening technique, the preventing shear failure of columns and the improving the ductility, the adjusting column stiffness and the making them unify as possible, or the increasing flexure capacity of columns may be aimed to improve the seismic performance of a building. Many retrofiting techniques are available and have been applied for such purposes.

9.1.1 R.C. Jacketing for Strengthening:

9.1.1.1 Calculation Techniques:

The Japanese techniques developed for calculations to improve ductility, to improve stiffness distribution, and to increase flexure capacity of a column as follows: [14,17]

i) <u>Calculation procedure for ductility improvement</u>:

(a) The existing condition on a column should be checked by using equation (9.5) if the ductility improvement would be possible for the column or not.

(b) Assuming the required Ductility Index F.

(c) Estimating the required ductility factor μ

corresponding to the F-value.

(d) Estimating the required shear reinforcement ratio p_{w2} corresponding to the required ductility factor μ.

(e) Checking the result if the target is achieved or not.

ii) Calculation procedure to improve stiffness distribution:

(a) Assuming the effective flexure length of a column after removal of non-structural spandrel walls.

(b) Estimating both the flexure and shear strength of the column using the effective flexure length, and also estimating both the values of Strength Index C and Ductility Index F.

(c) Checking the result if the target is achieved or not. If the target is not achieved, the combination with the other strengthening technique should be considered.

iii) Calculation procedure to increase flexure capacity:

(a) Estimating the values C and F required for the strengthening.

(b) Estimating the flexure capacity of a column corresponding to the value C, and estimation the required quantity of tension reinforcement corresponding to the flexure capacity.

(c) Estimating the shear reinforcement ratio p_{w2} corresponding to the value F.

(d) Checking the result if the target is achieved or not.

Where the notations mentioned in the above three calculation procedures for cases of strengthening a column, are given in the "STANDARD" [17] as follows:

C = Ultimate strength index of structure
(estimated by three level screening procedure; first, second, and third level procedures. The first level procedure is the simplest, but most conservative of the three, while the basic concept is common for all three) [17]

F = Ductility index of structure
(estimated by three level procedure and this index indicates grade of frame ductility)

μ = Ductility factor [$5 \geq \mu \geq 2$]

9.1.1.2 Calculation Equations:

The ultimate flexural strength a column may be computed by equation (9.1) which is given in the "STANDARD"

$$M_u = 0.8\ A_s \sigma_y h + 0.5Nh\ (1 - N/\ bhf_c) \quad \text{........(9.1)}$$

where, A_s= area of tension reinforcement, [mm^2]
b = width of compression face of column, [mm]
h = cross-sectional depth of column, [mm]
f_c= compression strength of existing concrete, [MPa]
N = the axial load of column [N]
σ_y= yield strength of longitudinal reinforcement [MPa]

The shear force of a column in which the yield hinges developed at both the top and bottom of a column may be computed by equation (9.2).

$$Q_{mu} = 2M_u/H_o \quad \text{........................}(9.2)$$

where, H_o= effective flexure length of column.

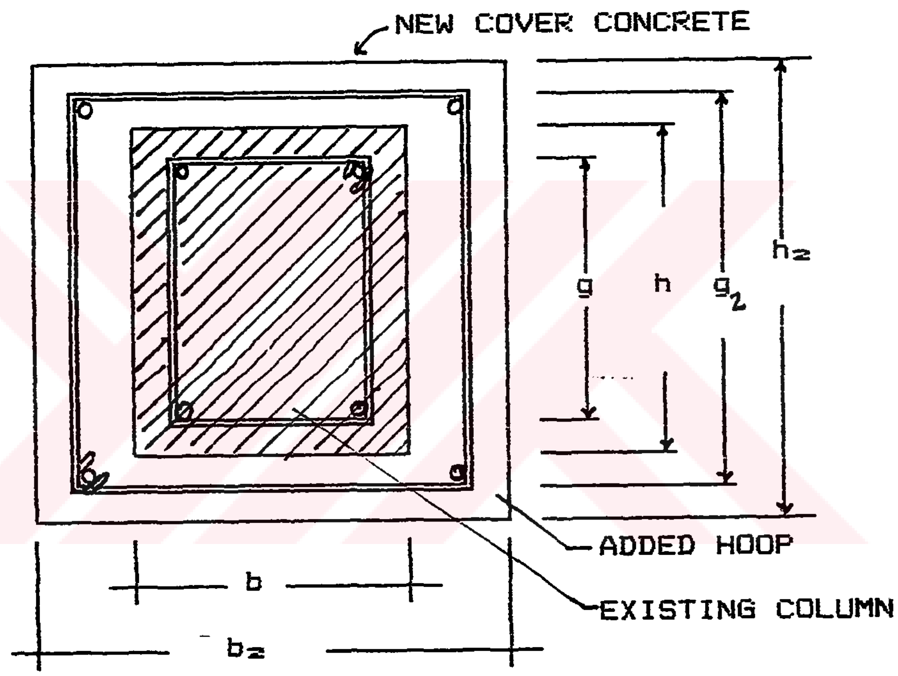


Fig. 9.1 Notations for columns after strengthening

The ultimate flexural strength of a column in which the new tension reinforcement are added may be computed by equation (9.3)

$$M_u = A_s\sigma_y\, g + A_{s2}.\sigma_{y2}\, g_2 + 0.5Nh_2(1 - N/b_2h_2f_{c1}) \quad .(9.3)$$

where, f_{c1} = compression strength of new concrete [MPa] the notations with the subscript 2 represent the notation after strengthening. See Fig. 9.1.

The ultimate shear strength of a column after strengthening may be computed by equation (9.4).

$$Q_{su} = \left\{ \frac{0.053 p_{t2}^{0.23} (17.658 + f_{c1})}{M/Qd_2 + 0.12} + 0.8457 \sqrt{p_w \sigma_{wy} + p_{w2} \sigma_{wy2}} + 0.1\,\sigma_o \right\} 0.8 b_2 h_2 \qquad \text{................(9.4)}$$

where, the notations with the subscript 2 represent the notation after strengthening. See Fig. 9.1, and,

$p_{t2} = A_8 / b_2 h_2$

p_w= shear reinforcement ratio in column.

σ_{wy}= yield strength of shear reinforcement in column [MPa]

p_{w2}= additional shear reinforcement ratio in column

σ_o= N/ bh [MPa]

The value M/Qd_2 may be taken as 1.0 if it becomes less than 1.0, and also may be taken as 3.0 if it becomes larger than 3.0

The value d_2 may be usually taken as $H_o/2$.

<u>Special Requirement to Keep Ductility:</u>

Columns in which the ductility improvement is required shall be within the following limitation;

$$70 p_t + \sigma_o < 37.5 H_o / (\alpha_1 h) \qquad \text{............(9.5)}$$

Where, the value α_1 usually may be taken as 1.0, p_t: original tension reinforcement ratio, σ_o: unit axial stress of co lumn by gravitational load, h_o: effective flexure length of column, h : depth of column.

Note: " all the requirement mentioned in Chapter 7 about columns have to be thoroughly considered and followed".

9.1.2 R.C. Jacket on Damage Columns:

After removal of the temporary load bearing supporting, loads are transmitted from the original column to the reinforced concrete jacket through the following mechanisms: [19]

a) Through bond and friction, developed between the original column and the jacket. Load is transferred through the jacket from the upper to the lower part of the column.

b) At the upper part of the column, load is transferred from the original reinforcement to the new one through the bent down bars (and vice versa at the lower part of the column, beyond the damaged region).

c) Concrete fills the region of damaged concrete contributing to partial restoration of column's continuity.

d) Directly from the beam to the top of the jacket, if pouring under pressure is used, from to the top of the column (i.e. through holes in the slab above).

e) Through the existing reinforcement, if buckling of the bars has not occurred.

Based on the above considerations some conservative rules for dimensioning of reinforced concrete jacket as follows: [2]

$$N_s \approx \left[\left(\frac{0.2}{l_o}\right) E_s + \alpha \left(\frac{N_{tot} - N_{res}}{A'_c}\right) \right] . A'_s \quad ..(9.6)$$

where:

N_s : load to be transferred by means of additional reinforcements (A'_s), welded on existing reinforcements, [N]

N_{tot}: total axial load to be retained to after taking-off temporary shores, and/or additional loads to act on the column after repair and/or strengthening, as well as after eventual shrinkage, creep redistribution, [N]

N_{res} : load to be trusted to the damage column (a roughly estimated percentage of N_{tot}), [N]

l_o : the half length of jacket [mm],

α = E_s/E_c,

A'_c : cross sectional area of additional concrete of jacket [mm^2]

A'_s: cross sectional area of longitudinal additional reinforcements. [mm^2]

Equation (9.6) based on the fact most of the friction is mobilized when concrete-to-concrete slip reaches a critical value of 0.2 mm.

If $\emptyset_h$ denotes the diameter of bars used for hoops provided in the jacket (every α_h) near its ends, the following equation secures the substitution of tensile strength of jacket's concrete by hoops"

$$f_{ct,max}.t.\alpha_h = f_{sy}.\pi\emptyset^2{}_h/4 \quad , \quad \text{or}$$

$$\alpha_h \approx \frac{2/3\ (f_{sy}/f_{ct}).\emptyset^2{}_h}{t} \qquad \text{...........(9.7)}$$

where: $f_{ct,max.}$ = maximum tensile strength of concrete [MPa]

The length v_o, along which these densely spaced hoops have to be provided, is estimated by means of the assumptions shown in Fig. 9.2 :

$$N_r \approx (\mu.\sigma_y).\ 8t. \quad \approx \mu\ f_{ct,min}.\ 8t. \quad , \text{ or}$$

$$v_o \approx 0.2\ N_r/\ \mu.f_{ct}.t \qquad \text{...............(9.8)}$$

where, N_r= load to be transferred to the section of the jacket (A'_c), by means of lateral friction (secured through horizontal compressive stresses produced by differential Poisson's deformation) [N]

μ = friction coefficient corresponding to

σ_c= $f_{ct,min}$

t = thickness of jacket. [mm]

Empirical formula is given for the dimensioning of the bent-down bars needed to connect existing reinforcement to the additional longitudinal reinforcements. This formula is valid when two levels of 4 bent-down bars are provided, above the damaged area: [2]

$$2/3\ (N_{tot} - N_{res}) = 4(0.1.\ \pi d^2/4 f_{sy} = 50.s_u d f_{cc})$$

where d = the diameter of the bent-down bars [mm]

s_u=critical slip (≈ 0.2 mm)

f_{sy} = yield stres of steel [MPa]

f_{cc} = cube concrete trength [MPa]

<u>Provisional recommendations:</u>

First of all, the increased -factors (for concrete and additional welded reinforcements) have to be taken into account, when dimensioning the R.C. jackets (Ref. 1 chapter 6). Also, additional requirements (in regard with minimum additional reinforcements, thicknesses of jackets, etc.) are summarized as follows:

1- In case of shotcrete:
   a) min thickness of jacket= 50 mm
   b) new welded re-bars, min. 1ø14/150
   c) densely spaced closed stirrups, min. ø8/100.
2- In case of cast-in-place concrete;
   a) min. thickness of jacket:
      for one cage of re-bars: 70-100 mm
      for two cages of re-bars: 100-150 mm
   b) for rebars and stirrups: the same requirements.

Also, the requirements for surface preparation and the problems of normal cast-in-place concrete as well as of shotcrete discussed in chapter 7 have to be thoroughly considered.

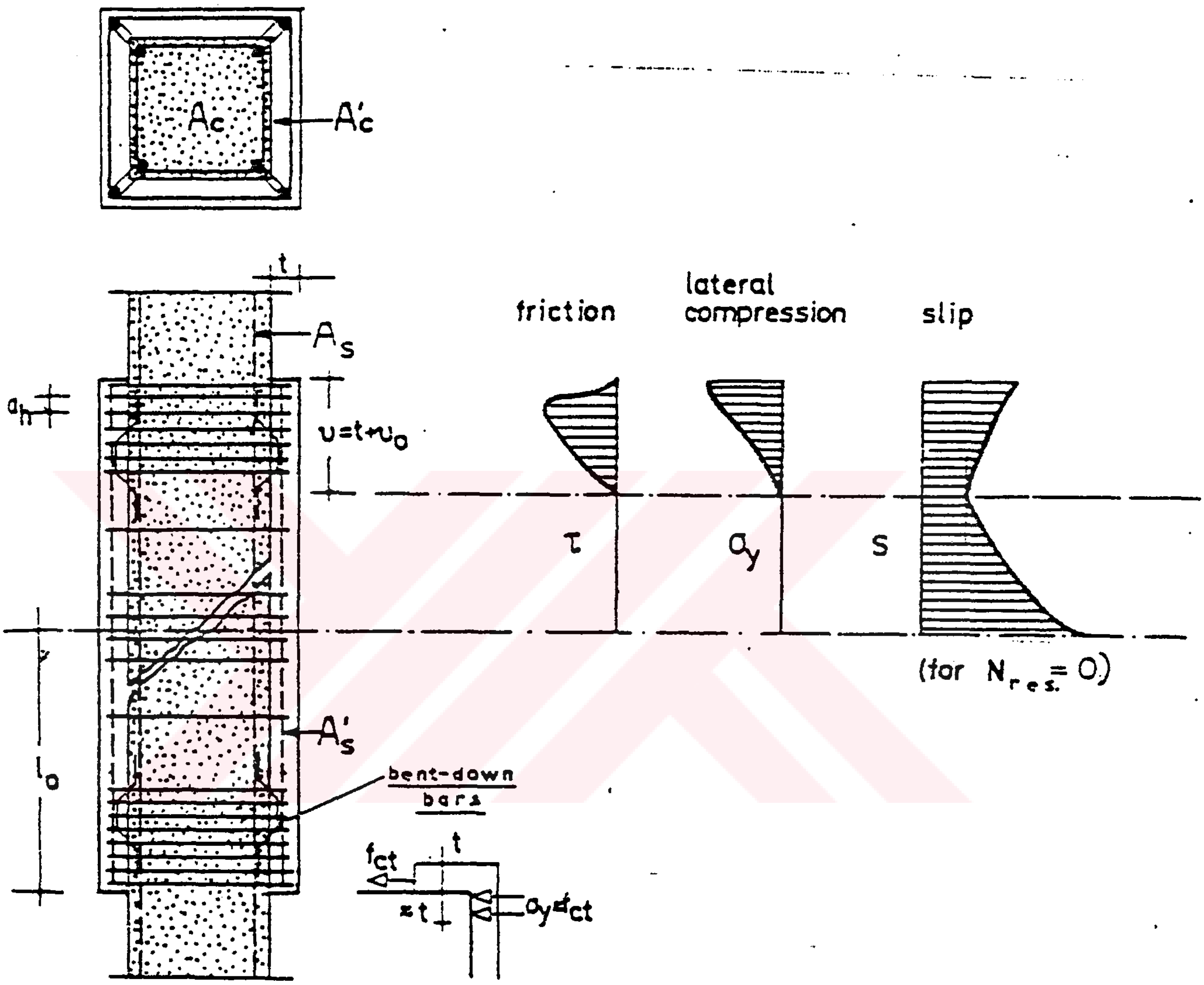


Fig. 9.2 R.C jacket for repair of columns: critical length v_o where jacket suffers the highest horizontal tension (Concentrated b friction transfer)

9.2 Beams:

The aim of repair and/or strengthening of beams is to provide adequate strength and stiffness of damaged beams which are deficient to resist gravity and seismic loads. It is important that the range of stiffness values for the beam and column be investigated for moment resisting frames subject to forces to insure that potential hinging will occur in beam rather in column. The beams must not be made too stiff with respect to the adjacent columns.

9.2.1 Flexural strengthening:

The available techniques for increasing the flexural capacity of a beam are, glued thin steel sheets and reinforced concrete jacket. Some experimental and theoretical data are vailed for these techniques. [2]

9.2.1.1 Glued thin steel sheets:

The decisive factor in dimensioning the steel plates to be bonded to the R.C. members (mainly beams), is the strength of their bonding. Steel strength cannot, therefore, be fully exploited. This is one of the reasons why thin plates are preferred ($t \leq 3mm$, suggested by some authors in case of ordinary building-elements), unless special mechanical anchorage systems are provided. [2]

Bond stresses in the anchorage area of steel glued on relatively thin concrete sections as follows:

$$\tau_a(x) = \sigma_{os} \cdot w \cdot t_s \cdot \frac{ch(w.x)}{sh(w.l_s)} \quad \ldots\ldots\ldots\ldots(9.9)$$

where,

σ_{os} = stress in the steel plate immediately ahead of the start of the anchorage zone in N/mm^2

$$w = \left[\frac{G_g}{t_g} \left(\frac{1}{E_s t_s} + \frac{1}{E_c t_c} \right) \right]^{1/2} \quad \text{in } mm^{-1}$$

G_g , t_g : shear modulus (in N/mm^2) and thickness (in mm) of the glue

E_c, t_c : modulus of elasticity (in N/mm^2) and thickness (in mm) of the concrete element

E_s, t_s : modulus of elasticity (in N/mm^2) and thickness (in mm) of the steel plate

x : position coordinate in mm

l_s : anchorage length in mm.

For continuously glued sheets, a full check of local bond stress may be needed at every point.

According to Greek "Recommendations for Repair/Strengthening" (N.T.U.,1978), the following formula for the local bond stress may be used, in case of continuously glued sheets:

$$\tau_{\alpha} = \frac{V}{b \cdot z_2 \cdot \left(1 + \dfrac{A_{s,1}}{A_{s,2}} \cdot \dfrac{d_1 - x}{d_2 - x} \cdot \dfrac{z_1}{z_2}\right)} \approx \frac{V}{b \cdot z_2 \cdot \left(1 + 0.85 \dfrac{A_{s,1}}{A_{s,2}}\right)} \qquad \text{..............(9.10)}$$

where:

V : shear force to be retained by the R.C. element After repair and/or strengthening. [N]

For the other notations are shown in Fig. 9.3.

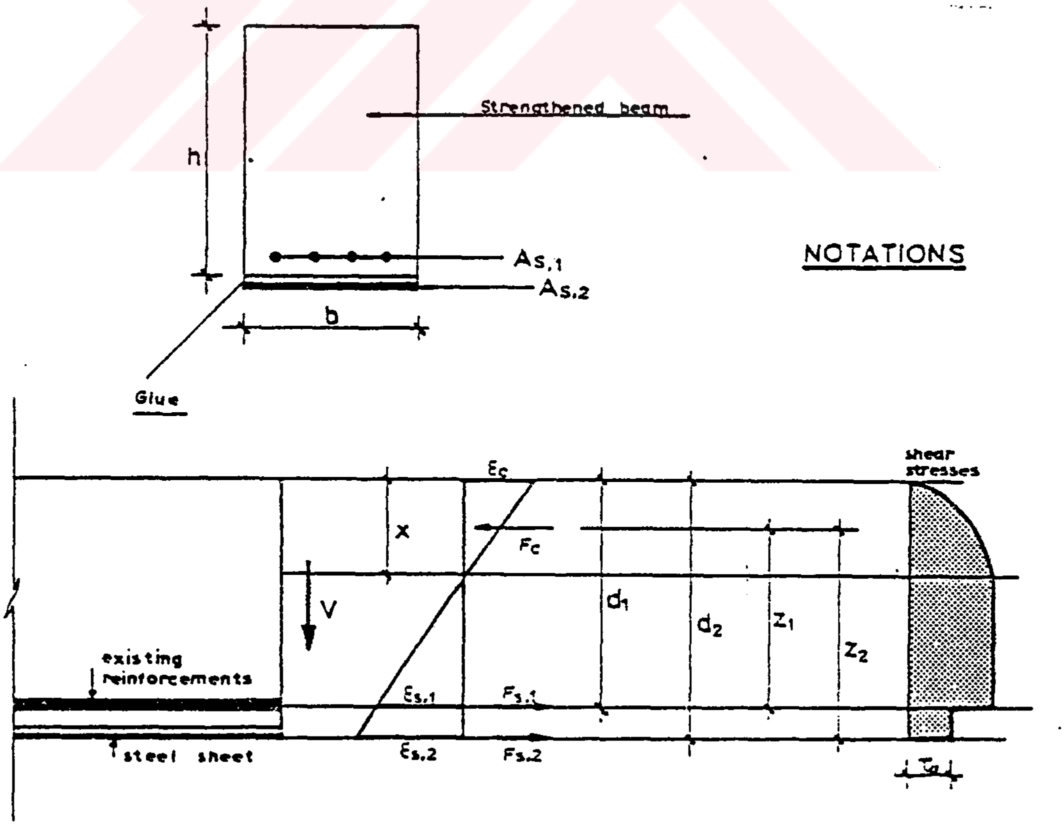


Fig. 9.3 Notation for equation (9.10)

Provisional recommendations:

1- Continuously glued sheets: The local bond strength should not be higher than the mean tensile strength of concrete foreseen by the C.E.B. -Model Code:

$$\tau_a \leq 1/\gamma_c . f_{ct,m} \qquad \ldots\ldots\ldots\ldots\ldots (9.11)$$

where: $f_{ct,m}$ = mean value of tensile strength of concrete

2- Sheets glued only at anchorage zones: In such a case, on the basis of the actual stage of knowledge, two possible approaches may be used in calculating anchorage lengths (although a considerable uncertainty should be expected in this connection):

i) check of the local bond stress $\tau_{a,max}$ for a chosen anchorage length l_a, as explained before. However, due to the very high τ_a-values calculated with this method, it is temporarily suggested to introduce a rough correction factor equal to 2 (instead of 3):

$$1/2\tau_{a,max} \leq (1/\gamma_c).f_{ct,m}. \qquad \ldots\ldots\ldots\ldots (9.12)$$

ii) A more pragmatic approach is based on pull-out force-displacement diagrams Experientially or theoretically prepared, taking also into account the time-effects, see the Fig. 9.4.

First, these diagrams have to be "reduced" (dividing their ordinates by a γ_m-factor, say equal to 1.5, in order to asses a "safe" behavior).

-If the full bearing capacity of such an anchorage is to be exploited, then an anchorage length l_{a1} is found in the figure for a design force $F_{a,d}$.

-If a limited front slip S_d is imposed by design conditions (acceptable deflections or other serviceability requirements), then a larger anchorage length l_{a2} is found in the figure for a design force $F_{s,d}$.

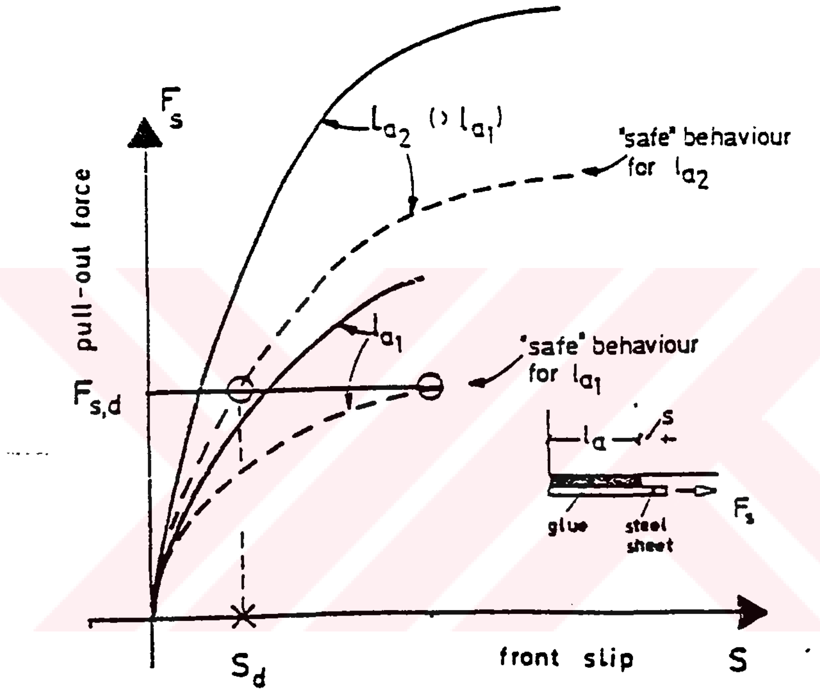


Fig. 9.4 Relation-ship between the pull-out force and front slip

However, additional mechanical anchorages (e.g. by means of dowels) are recommended for the anchorage zones of such steel sheets in any case. [2]

It is also recommended that the steel sheet section be greater, by at least a factor 1.5, than the section calculated on the basis of the strength of steel:

$A_s \approx 1.5.A_{s,needed}$, ...(9.13)

on the γ_m-factor for the additional steel has to be grater, by a factor 1.5, than the normal factor for existing material:

$\gamma_s' \approx 1.5\,\gamma_s$

when a section is strengthened by use of bonded steel sheets, additional bending moment capacity must not exceed half of that the section before any repair and/or strengthening works:

$\Delta M_{u.add.} \leq 1/2(M_{u,exist.})$

Finally, the requirements mentioned in chapter 7 (in regard with thickness of sheets and bond layers, surface preparation, protection against fire, etc.) have to be thoroughly considered and followed.

9.2.1.2 Reinforced Concrete Jackets:
(cast-in-place concrete or shotcrete)

Dimensioning of a beam strengthened by R.C. jacket on top and/or on bottom of the beam (cast-in-place concrete or shotcrete plus additional welded re-bars), have been presented in C.E.B. techniques [2] in form of -factors, because redimensioning may be carried out taking into account the entire section constituted both of the old and the additional materials, as if the final section were monolithic. Obviously, such a hypothesis leads to strength and stiffness characteristics ($M_{u,m}$, $V_{u,m}$, K_m) higher than the real ones ($M_{u,r}$, $V_{u,r}$, K_r). Corresponding correction factors are foreseen by the Greek "Recommendations for Repair/Strengthening", as follows:

a) <u>Cast-in-place concrete on the top of the building element:</u>

When $\Delta A_c \leq 1/3\ A_c$ (A_c: area of existing R.C. element)

for beams: $M_{u,r} : M_{u,m} = 0.80$

$V_{u,r} : V_{u,m} = 0.80$

$K_r : K_m = 0.65$

When $\Delta A_c \geq 1/3 A_c$ (A_c: area of existing R.C. element)

for beams: $M_{u,r} : M_{u,m} = 0.65$

$V_{u,r} : V_{u,m} = 0.65$

$K_r : K_m = 0.40$

b) <u>Shotcrete</u> (gunite)

(plus additional welded reinforcements)

for beams: $M_{u,r} : M_{u,m} = 0.80$

$V_{u,r} : V_{u,m} = 0.80$

$K_r : K_m = 0.65$

Where:

$M_{u,r}$ = the real bending strength of the final section,

$M_{u,m}$ = the bending strength as if the final section were monolithic.

$V_{u,r}$ = the real shear strength of the final section

$V_{u,m}$ = the shear strength as if the final section were monolithic.

K_r = the stiffness of the final section

K_m = the stiffness as if the final section were monolithic

Provisional recommendations:

i) When the provisions presented above are observed, it seems that no major problem arises regarding the dimension of these R.C. jackets.

The max. shear stress due to bond is recommended to keep it lower than two thirds of the mean tensile strength of the "old" concrete: [2]

$$\tau_{\alpha,max} \le \frac{1}{\gamma_c} \cdot \frac{2}{3} \cdot f_{ct,m}$$

$$\approx \frac{1}{\gamma_c} \cdot \frac{2}{10} \; f^{2/3}_{ck} \qquad \ldots\ldots\ldots(9.14)$$

where τ_α , f_c in MPa.

The maximum bond stress at the interface of flexural elements can estimated by equation (9.10). Nominal strength can be increased, in case shear connectors are used, by an amount of approx. 50% for closely spaced anchors if no more advanced redimensioning methods are used. [2]

-For flexural elements repaired/strengthened by shotcrete jackets, the max. shear stress due to bond $\tau_{\alpha,max}$ is recommended to be less than the one fourth of the mean tensile strength of the "old" concrete; it is recommended that in any case, a minimum of shear connectors (capable of transferring a stress equal to $0.4\tau_{\alpha,max}$) should be provided. In case flexural reversals or dynamic loading are expected, the whole shear force at the interface is recommended to be transferred through shear connectors and, anyhow, the maximum shear stresses has to be less than the two thirds of the mean tensile strength of the "old" concrete. [2]

ii) It is recommend that γ_m-factors for concreteand additional welded reinforcement be greater. It is generally recommended that the thickness of the new concrete layer has to be less than the thickness of the existing concrete.

$$\Delta A_c \leq 1/3\ A_c.$$

9.2.2 Shear Strengthening:

The available techniques to increase the shear capacity of a damage and/or weak beams are glued thin steel sheets, and R.C. jackets.

9.2.2.1 Glued Thin Steel Sheets:

The decisive factor in dimensioning the steel plates to be bonded to the R.C. beams, is the strength of their bonding. Steel strength cannot, therefore be fully exploited. In case of ordinary building-elements the thickness of the plate must be less or equal 3mm, unless special anchorage systems are provided.

Fig. 9.5 show the oversimplified model for calculations of this techniques, [2] the diagonal cross-sections of the glued steel sheet (limited to an inclined shear crack) is undertaking the shear force V_s

$V_s = V - V_{exist.}$ ....(9.15)

where:

V = shear force to be retained by the R.C. element after repair and/or strengthening. [N]

$V_{exist.}$= shear force to be trusted to the existing (damaged and/or weak) R.C. element. [N]

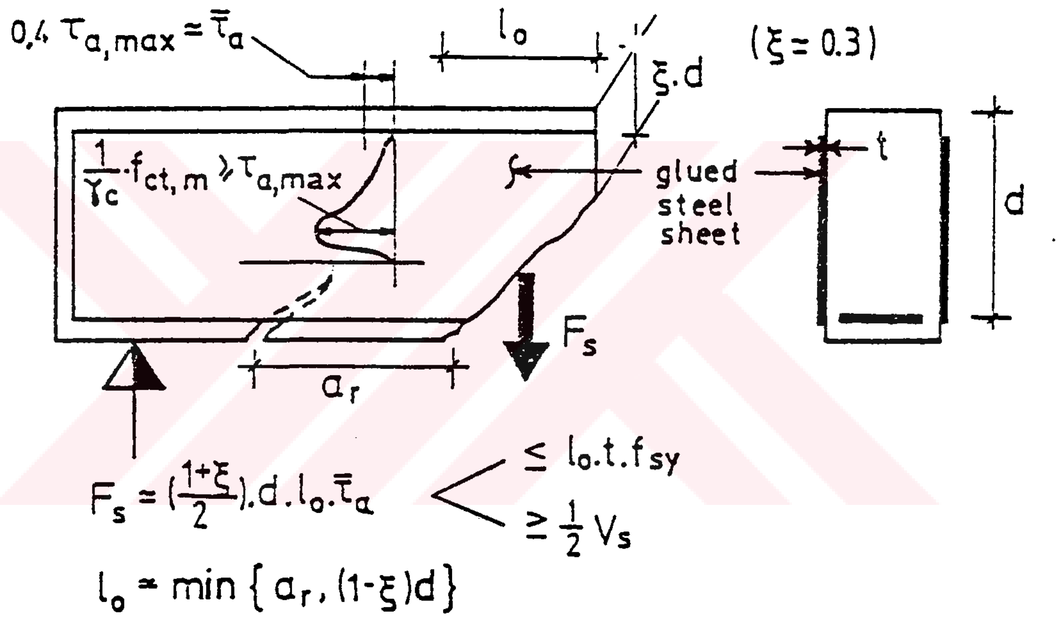


Fig. 9.5 Oversimplified model for calculation of shear force of beam, strengthening by glued thin steel sheets

For design purposes, a rough approximation of the mean value τ_a of the bond stresses could be sought, e.g. $\tau_a \approx 0.4\ \tau_{a,max.}$, so that the thickness t of the sheet and the bond strength $\tau_{a,max}$needed should fulfill the following conditions (which, of course, must be completed by the appropriate partial safety factors):

$$t \geq 0.25\, d.\, \frac{\tau_{\alpha,max}}{f_{sy}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots(9.16)$$

$$Vs \leq \frac{d.l_o}{2} \cdot \tau_{\alpha,max.} \qquad \ldots\ldots\ldots\ldots\ldots\ldots(9.17)$$

where the notation are shown in Fig. 9.5.

<u>Provisional</u> recommendations: [2]

i) This method of glüing thin steel sheets used for special cases , when the advanced technology and high level of quality control required are available.

ii) According to C.E.B.-model code the maximum shear stress due to bonding to be less than the mean tensile strength of the concrete;

$$\tau_{\alpha,max.} \leq \frac{1}{\gamma_c} \cdot f_{ct.m} \approx \frac{1}{\gamma_c} \frac{3}{10} \cdot f_{ck}^{2/3} \quad \ldots(9.18)$$

where τ_{α}, f_c in MPa.

iii) If the condition regarding the shear force which can be transferred ($V_s \leq dl_o/2\ \tau_{\alpha,max.}$) is not fulfilled, appropriate addition mechanical connectors should be used.

iv) It is also recommended that the steel plate section be greater, by at least a factor 1.5, than the section calculated on the basis of the strength of steel:

$$A_s \approx 1.5\ A_{s.needed} \qquad \text{.................(9.19)}$$

v) When a section is strengthened by use of bonded steel sheets, additional shear capacity must not exceed half of the section before any repair and/or strengthening works:

$$V_{u,add} \leq 1/2\ V_{u,exist.} \qquad \text{.............(9.20)}$$

Also, the requirements mentioned in chapter 7 and 8 have to be thoroughly considered and followed.

9.2.2.2 Reinforced Concrete Jackets:

According to the extensive experimental and theoretical investigation summarized in reference [2] which have been presented in a form of factors as follows:

$$V_{u,r} \approx 0.80\ V_{u,m} \qquad \text{............ ...(9.21)}$$

where:

$V_{u,r}$ = the real shear strength of the final section; [N]

$V_{u,m}$ = the shear strength as if the final [N] section were monolithic.

Fig. 9.6 show oversimplified model for calculation of shear force (V_s) taken by the additional tensioned stirrups,

$$V_s = V - V_{exist.} \qquad \text{....................(9.22)}$$

Where:

V = shear force to be retained by the R.C. element after repair and/or strengthening. [N]

$V_{exist.}$ = shear force to be trusted to the existing (damaged and/or weak) R.C. element. [N]

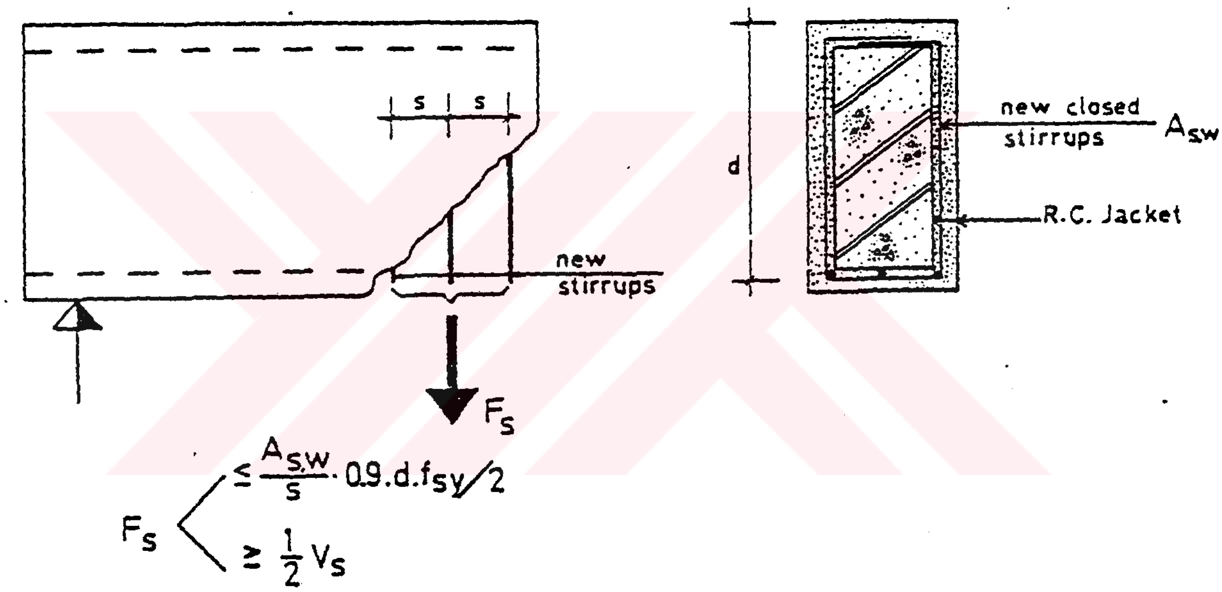


Fig. 9.6 Oversimplified model for calculation of shear force taking by additional stirrups

For design purposes the amount of new stirrups should fulfill the following condition (which, of course, must be completed by the appropriate partial safety factors):

$$A_{sw}/s \geq \frac{1/2\ V_s}{0.9d.\ f_{sy}/2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(9.23)$$

where the steel stress at limit state has to be taken as equal to half of its yield strength, in order to account for deviations of additional stirrups when they are applied on a round irregular periphery of a concrete element.

In cases of non-closed stirrups or diagonal bars for shear strengthening, the safe transfer of forces acting on these additional reinforcements has to be secured by adhesion and additional mechanical heavy duty connectors (dowels, anchors, etc.)

Provisional recommendations:

i) It is recommended, that γ_m-factors for concrete and additional welded reinforcements be greater.

ii) When R.C. beams are strengthened by R.C. jackets, it is generally recommended that the thickness of the new concrete layer be less than one third the thickness of the existing concrete:

$$\Delta A_c \geq 1/3\ A_c \qquad \text{.......................(9.24)}$$

iii) All the requirements mentioned in chapter 7 and 8 be thoroughly considered.

9.3 In-filled Shear wall:

An extensive amount of data of theoretical and, mainly, experimental investigations of the behavior of strengthened R.C frames (damaged and/or weak) by means of infilling shear wall have been reported in the literature. [20-30]

The in-filling shear walls includes: (1) to fill the existing frame by a new reinforced concrete shear wall. (2) to fill the opening of existing reinforced concrete wall with new concrete and to make a monolithic wall. and (3) to increase the original wall thickness with new concrete. The new reinforcement concrete shear walls are anchored to the existing R.C. frames or walls by the post-installed anchors or the concrete shear keys or welding the rebars in the existing frame. This technique increase the shear strength of frame. However, the flexural strength or the strength which is dominated by the uplift of foundation may become lower than the shear strength. There fore, the smallest of the shear strength, flexural strength or the foundation uplifting strength should be taken as the final strength after strengthening. [14]

The building strengthening by in-filling shear walls generally should be designed by the strength design concept. However, the building may be strengthened by the ductility design concept. If the member performance of in-filled shear wall will be dominated by the flexural failure of in-filled wall or the uplifting of foundation.

The unit shear stress of $0.25f_c$ (f_c: compressive strength of concrete) for the horizontal cross-sectional area of in-filled wall may be expected to estimate the required cross-sectional area of in-filling shear wall without opening. The following values may be assumed for the ductility index F of in-filled shear wall. [14]

F = 1.0 for case of shear failure of shear wall.
F = 2.0 for case of flexural failure of shear wall.
F = 3.0 for case of uplifting of foundation.

9.3.1 Calculation Techniques:

9.3.1.1 Calculation Procedure:

The calculation procedure for in-filling shear walls is as follows: [14]

1- Evaluate the structural performance of the existing building.
2- Determine the strengthening policy, for example, the strength design concept, ductility design concept, or its combination.
3- Determine the strength or ductility required for the building.
4- Assume the compressive strength of concrete for strengthening, and assume the unit design shear stress of in-filling wall.
5- Assume the thickness of in-filing wall, to estimate the wall length required, and to decide the arrangement of walls.
6- Calculate the shear reinforcement ratio of wall,

and estimate the number of post-installed anchors required.

7- Calculate the maximum shear strength of in-filled shear wall after strengthening.

8- Check if the total seismic performance of the entire building reached the target of strengthening or not. If it did not reach the target, the procedure from the step (5) should be required again.

9.3.1.2 Calculation Equations:

The shear strength which would be expected for an in-filled shear wall should be taken the smallest of the following: [14]

1- 80 thorough 90 percent of the shear strength which may be calculated assuming that the in-filled wall is jointed monolithically with the boundary columns and beams, such as an I-section member. However, 90 through 100 percent of the shear strength as a monolithic shear wall be taken, if the post-installed anchors will be provided for the joint.

2- The shear strength which may be calculated assuming the special shear transfer mechanism.

3- The strength which is dominated by flexural failure.

4- The strength which is dominated by uplifting of foundation.

The following equations may be used to calculate the capacities of the above.

i) <u>Shear strength as a monolithic shear walls</u>:

The equation which is given in the "Standard for Evaluation of Seismic Capacity of Reinforced Concrete Buildings" [17] may be used to calculate the shear strength as a monolithic shear wall but some of the notation redefined as follows:

$$ {}_{w}Q_{su} = \left\{ \frac{0.053 p_t^{0.23}(17.658+f_c)}{M/QL + 0.12} + 0.8457\sqrt{p_w \, \sigma_{wy}} + 0.1\sigma_o \right\} b_e j \qquad \text{...........(9.25)} $$

Where:

$p_t = 100A_s/b_eL$

A_s = total area of longitudinal reinforcement in column at tension side. [mm^2]

$b_e = A/L$

A = total cross-sectional area of wall included boundary columns. [mm^2]

L = distance from extreme compression fiber to extreme to tension fiber. [mm]

$p_w = A_w/b_e.s$

A_w = area of horizontal reinforcement in wall. [mm^2]

s = spacing of horizontal reinforcement in wall. [mm]

σ_{wy} = yield strength of vertical reinforcement in wall. [MPa]

$\sigma_o = N / b_e.L$ [MPa]

N = total gravitational loads on boundary columns. [N]

M/Q = ratio of moment to shear

[where, $Q = {}_{w}Q_{mu} = 2\ {}_{w}M_u / H_w$, and,

${}_{w}M_u = A_g\ \sigma_y\ l_w + o.5\ A_w.\sigma_{wy}.l_w + o.5Nl_w$]

j = the smaller of 0.8L or distance between centroid of boundary columns. [mm]

l_w = distance between both centroids of boundary columns [mm]

A_g = total area of longitudinal reinforcement in boundary column at tension side, [mm^2]

ii) Shear strength by special shear transfer mechanism:

The shear strength should be the smaller of the following:

$${}_wQ_{su} = {}_wQ'_{su} + 2\ \alpha\ Q_c \qquad \text{...............(9.26)}$$

$${}_wQ_{su} = Q_j + {}_pQ_c + \alpha\ Q_c \qquad \text{..............(9.27)}$$

where:

${}_wQ'_{su}$ = shear strength of in-filled wall panel.

Q_j = shear strength of post-installed anchors which are arranged to the top and bottom of in-filled wall.

${}_pQ_c$ = punching shear strength of the one side of boundary columns.

Q_c = the smaller of flexural strength or shear strength of the other side of boundary columns.

α = deformation compatibility factors, 1.0 for shear failure and 0.7 for flexural failure.

iii) Flexural strength as an I-section member:

The equation which is given as equation (14) in the "Standard for Evaluation of Seismic Capacity of Reinforced Concrete Building" [17] may be used to calculate the flexural strength of the wall, but the strength transferred by the reinforcement in the wall which equal to (0.5 A_w σ_{wy} l_w) should be replaced by the strength of the post-installed anchors and became as follows:

$$ {}_{w}M_{u} = A_{g}\ \sigma_{y}\ l_{w} + Q_{j} + 0.5Nl_{w} \qquad \text{.........}(9.28) $$

Where:

A_g = total area of longitudinal reinforcement in boundary column at tension side. [mm²]

σ_y = yield strength of longitudinal reinforcement of column. [MPa]

l_w = distance between centroid of boundary columns, [mm]

Q_j = shear strength of post-installed anchors. [N]

N = total gravitational loads of boundary columns. [N]

Also the strength transferred by the reinforcement in the wall should be neglected in case shear keys is used to joint the shear wall to the column, and the became as follows:

$$ {}_{w}M_{u} = A_{g}\ \sigma_{y}\ l_{w} + 0.5Nl_{w} \qquad \text{..................}(9.29) $$

iv) <u>Punching shear strength of boundary column:</u>

$$ {}_{p}Q_{c} = K_{min}\ \tau_{o}\ b_{e}\ h \qquad \text{.................}\ (9.30) $$

Where:

$K_{min} = 0.34 / (0.52 + a / h)$

$\tau_o = 10 + 0.1\ f_{c1} + 0.85\ \sigma$ [MPa]

for $0 < \sigma < (0.33\ f_{c1} - 28)$

and, $\tau_o = 0.22\ f_{c1} + 0.49\ \sigma$ [MPa]

for $(0.33\ f_{c1} - 28) < \sigma < 0.66\ f_{c1}$

b_e = effective width of the column which is punched. [mm]

h = depth of the column which is punched. [cm]

$a = h/3$ [mm]

f_{c1} = compressive strength of the existing frame. [MPA]

$\sigma = p_g.\sigma_y + \sigma_o$ [MPa]

$p_g = A_g / b_e.h$

A_g = total area of the longitudinal reinforcement of boundary column. [mm^2]

σ_y = yield strength of the longitudinal reinforcement of boundary column. [MPa]

$\sigma_o = N / b_e.h$ [MPa]

N = axial force of the boundary column. [N]

9.3.1.3 Design of In-filled Wall Panel:

1- First the Ductility Index F should be determined, and design shear force Q_D for the in-filled wall panel should be decided. [14]

a) The design shear force for the in-filled wall by using the post-installed anchors approximately as follows:

$$Q_D \approx \phi\ {}_wQ_{au} \qquad \ldots\ldots\ldots\ldots\ldots(9.31)$$

where:

${}_wQ_{au}$ calculated by equation (9.25)

In case of the embedded length of the anchor equal $5d_a$:

$\phi = 0.8$ anchors arranged to only top and bottom of a wall

$\phi = 0.9$ anchors arranged to the surroundings of a wall

In case of the embedded length of the anchors equal $8d_a$

$\phi = 0.9$ anchors arranged to only top and bottom of a wall

$\phi = 1.0$ anchors arranged to the surroundings of a wall

2- The unit shear strength τ_w computed by equation (9.32) should be less than the upper limit τ_D which would depend on the Ductility Index F assumed.

$$\tau_w = Q_D / t_w . l_w$$

where, t_w = thickness of in-filled wall.
l_w = clear length of the in-filled wall.

The upper limit of unit shear strength τ_D are given as follows:

$\tau_D = 0.16\ f_c$ [for F = 3.0]
$= 0.20\ f_c$ [for F = 2.0]
$= 0.25\ f_c$ [for F = 1.0]

3- The shear reinforcement of the in-filled wall should be provided to satisfy the following equation

$$\beta . Q_{wu} > Q_D \qquad \text{.....................(9.32)}$$

$\beta = 0.9 \div 1.0$ [in case of that the post-installed anchors are arranged to the surrounding of in-filled wall panel]

$\beta = 0.8 \div 0.9$ [in other cases]

where, Q_{wu} may be computed by the equation which is given as equation (15) in the "Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings" [17]

9.4 Steel Bracing Systems:

As mentioned in chapter (7) section (7.7.3) steel bracing systems, the dimensioning and the design method mentioned hereafter will be for the following techniques: [14]

1- The technique using the special steel frame to set the steel braces or panels into the existing reinforced concrete frame.

2- The technique using joint mortar or concrete and the post-installed anchors to fix the steel braces, called as the INDIRECT JOINT TECHNIQUE.

9.4.1 Target for Seismic Performance:

i) <u>Resisting mechanism depending upon steel bracing system:</u>

The structure after strengthening is assembled by three resisting components as follows:

a) Existing reinforced concrete frame.

b) Steel bracing system.

c) Connections between the existing reinforced concrete frame after the in-filled steel bracing system.

And the structural performances after strengthening , and they are summarized in table 9.1. The Type-I structural performance listed in Table-9.1

is the most resisting system after strengthening.

Table-9.1 : Resting System after Strengthening by Steel Bracing System

Types	Resisting Mechanism or Failure Pattern		
	R.C. Existing Frame	Steel Bracing System	Connection both
I	columns/beams shear or flexure yield	Braces or panels tension or shear yield	Don't reach its maximum strength
II	Columns or beams shear failure	Braces or panels don't reach their strength	Joint mortar or concrete slip
III	Tension side column yields by tension	Don't reach their strength	Don't reach its maximum strength
IV	Short columns shear failure	Braces or panels tension or shear yield	Don;t reach its maximum strength

ii) <u>Another resisting mechanism</u>:

Another mechanism, such as foundation uplifting, may be allowed after strengthening, because of that frame deformation by the foundation uplifting may dissipate an vibration energy without any damage on the frame or steel bracing systems. [14]

iii) <u>Decision of resisting mechanism for design</u>:

The total story shear adding the share of steel bracing system should be estimated for each mechanism in Table 9.1 including foundation uplifting mechanism. The minimum among them may be adopted as the lateral load for the design of steel bracing system. The share of story shear by the reinforced concrete frame can be estimated by "The Standard for Evaluation of Existing Reinforced Concrete Buildings". [17]

iv) Ductility index F after strengthening:
The value of Ductility index F in each type after strengthening should be followed in Table-9.2

Table-9.2: Ductility Index F after Strengthening

Types	Ductility Index F
I	The value that is estimated for the existing reinforced concrete frame, but $2.0 < F < 3.0$
II	$F = 1.5$
III	$F = 2.0$
IV	$F = 1.27$

For the foundation uplifting mechanism, the value 3.0 may be taken as the ductility Index F.

9.4.2 The Indirect Joint Technique:

The indirect joint technique is actually recommended for the application of strengthening by steel bracing systems. **Fig. 9.7** and **Fig. 9.8** show the details of this joint. The tests should be done to confirm the performance of joint concrete before an actual application, if the different details from the figure shown are applied. The clearance h' between the existing frame and the steel frame should be I60 mm through 250 mm. to anchor the stud bolts and to keep the lap length L in the joint concrete. The thickness of the plate of steel frame, t, should be more than $d_s/4$, (d_s : diameter of stud bolt). [14]

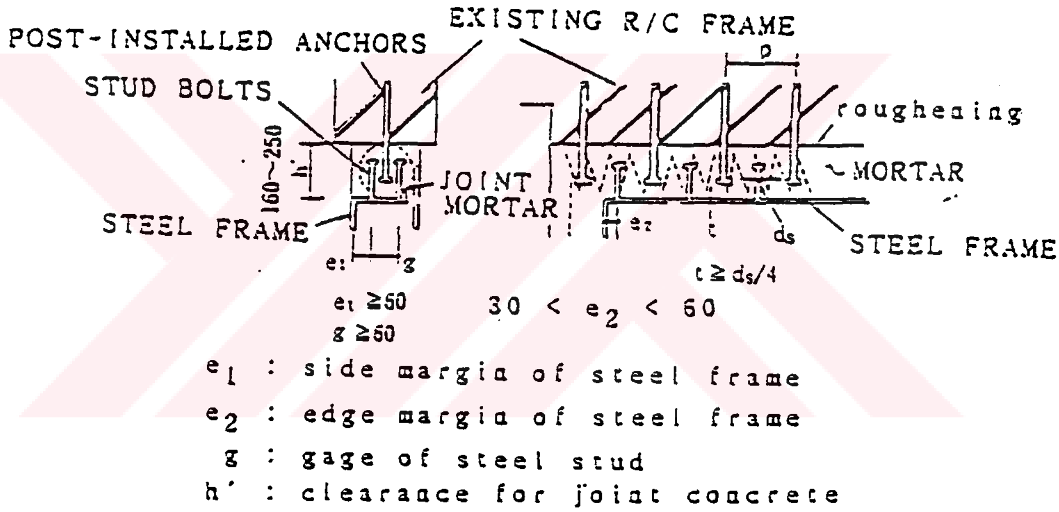


Fig. 9.7 Details recommended for joint concrete area

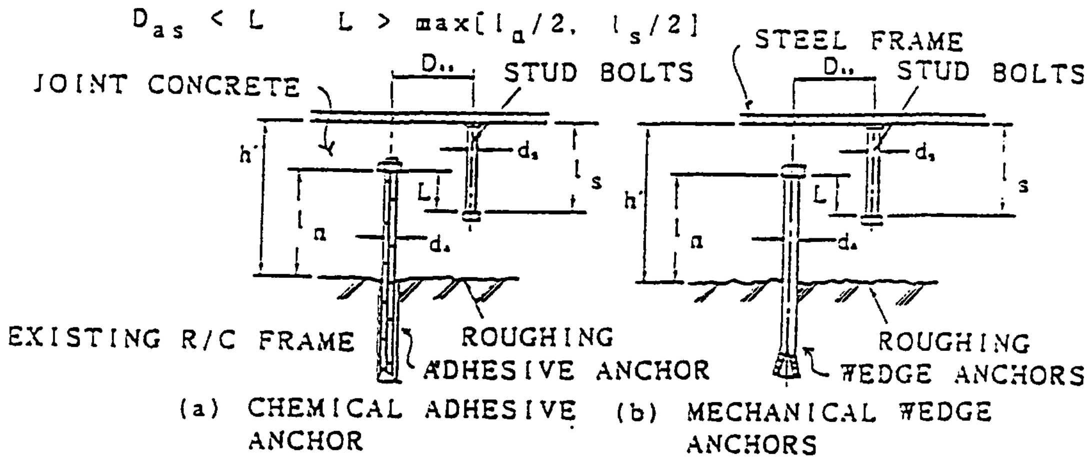


Fig. 9.8 Details recommended for joint concrete area

The anchor should have a head as shown in Fig. 9.9 in case using the mechanical wedge anchor without a splicing rebar.

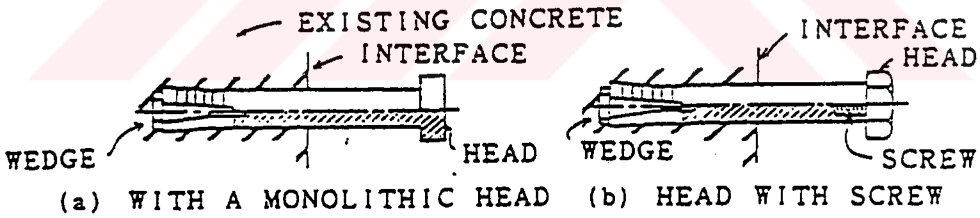


Fig. 9.9 Mechanical wedge anchor with head

The details to reinforce the joint concrete by using spiral wires, hoop shape reinforcement, or welded wire fabrics are shown in Fig. 9.10. In case of spiral wires, the diameter of wire should be more than 6 mm, and the pitch should be less than 40 mm through 60 mm. In case of hoop shape or welded wore fabrics, the diameter should be more than 10 mm, and the pitch should be less than 1/2 through 1/3 of the steel stud pitch. The reinforcement ratio in the joint concrete should be at least 0.4 percent to confine the concrete, to prevent the splitting of concrete. [14]

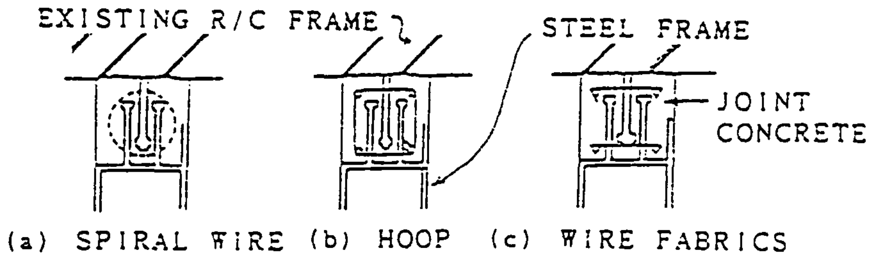


Fig. 9.10 Reinforcement in joint concrete area

9.4.3 Calculation Techniques: [14]

i) <u>General assumptions</u>:

1- The seismic capacity of steel bracing system should be the minimum among the five resisting mechanisms which were shown in Table-9.1 including the foundation uplifting mechanism.

2- The strength of existing reinforced concrete member should be calculated using the original member size.

3- It may be assumed that both the tension and compression brace reach the maximum strength at the same time. At the time, however, the unit compression stress in the brace member, f_{cr}, shall not exceed:

$$f_{cr} = \{ 1 - 0.4 (\lambda/\Lambda)^2 \} F \qquad \text{for } \lambda < \Lambda$$

$$= 0.6 F / (\lambda/\Lambda)^2 \qquad \text{for } \lambda > \Lambda \quad \text{.........(9.33)}$$

where,

λ = effective slenderness ratio

Λ = limit slenderness ratio (= $\pi^2 E / (0.6F)$

F = yield point strength of steel.

E = Young's modulus of steel

4- In case of steel panel system, the shear yielding of steel plate should be assumed to calculate the strength of the system, and the necessary steel stiffener should be arranged to prevent the buckling of panel in its elastic rang.

ii) Design procedure for strengthening by steel braces members:

1- Deciding the share of lateral load by the steel brace system.

2- Assuming the size of a steel frame or braces.

3- Proportioning steel stud bolts or post-installed anchors. The unit strength of steel stud bolts, q_{ds} may be calculated by:

$$q_{ds} = 0.64\ \sigma_{max}\ A_{ds} \qquad \text{................(9.34)}$$

where, σ_{max} = tension strength of stud bolt, should be less than 4100 kg/cm^2.

A_{ds} = area of stud bolt.

4- Proportioning the joint between the steel frame and the braces

iii) Design procedure for strengthening by steel panels:

1- Assuming the size of joint mortar between the steel panel and the existing reinforced concrete frame.

2- Assuming the size of opening or window within a panel.

3- Deciding the share of lateral load by steel panel system.

4- Proportioning the thickness of panel under the shear yield assumption.

5- Proportioning the flange plates or stiffener not to yield by ension or not to make buckling of a panel.

iv) <u>Calculation equations</u>:

The potential lateral load capacity of a frame after strengthening by the steel bracing systems, ${}_sQ_{su}$, may be taken as the smaller of the values which are calculated by equations (9.35) or (9.36).

$$ {}_sQ_{su} = {}_sQ_u + Q_{c1} + Q_{c2} \qquad \ldots\ldots\ldots\ldots\ldots (9.35)$$

$$ = Q_j + {}_pQ_c + Q_{c2} \qquad \ldots\ldots\ldots\ldots\ldots (9.36)$$

where, ${}_sQ_u$ = capacity that is dominated by steel bracing system, such as the strength of steel braces or steel panel which are calculated by equation (9.37) and (9.38). The others, Q_{c1}, Q_{c2}, Q_j, and ${}_pQ_c$, may be calculated by using the equations which are given for adding shear walls. See equation (9.26)

The later load that is transferred by the X-shape braces may be calculated by equation (9.37)

$$ {}_sQ_u = (N_c + N_t)\cos\theta \qquad \ldots\ldots\ldots\ldots\ldots (9.37)$$

where, the compressive strength N_c should be the smaller of compression yield strength or buckling strength, and the tension strength N_t may be computed under its yield stress level. The angle θ is shown in **Fig. 9.11.**

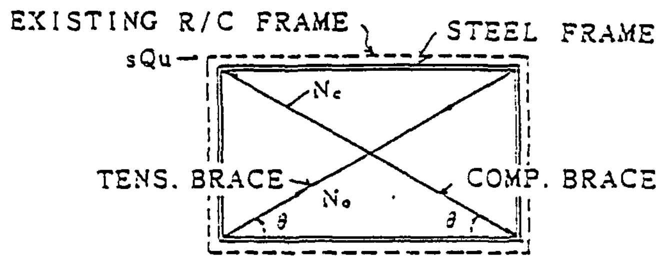


Fig. 9.11 Notation and assumption for load transfer by steel braces

The lateral load that is transferred by the steel panel may be calculated by equation (9.38)

$$_{s}Q_u = \tau_y .\ t .\ l_e \quad \text{.......................(9.38)}$$

where, τ_y = shear yield stress of steel plate

$= \sigma_y / \sqrt{3}$

t = thickness of panel.

l_e = length of panel, effective length without opening, see Fig. 9.12.

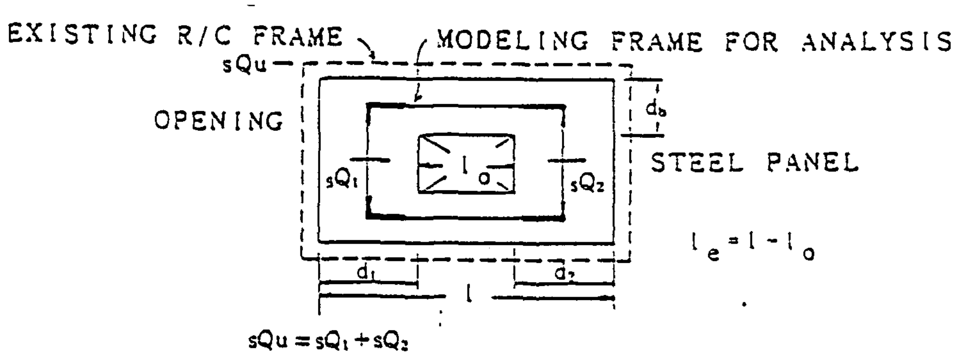


Fig. 9.12 Notation and idealization for load transfer by steel panel

In case of the panel with opening, the flexural strength of the region which correspond to beams or columns of a panel shown in Fig. 9.12 should be checked by using equation (9.39) and (9.40), and the thickness of flange plate should be decided not to yield by flexure.

$${}_bM_y = \sigma_y \, . \, A_f . \, d_b \qquad \text{for beam region} \quad ....(9.39)$$

$${}_cM_y = \sigma_y \, . \, A_f . \, d_c \qquad \text{for column region} \quad ....(9.40)$$

where, σ_y = yield strength of steel used.

A_f = area of flange of beam or column regions.

d_b = depth of beam region.

d_c = depth of column region.

See Fig. 9.12

The capacity of lateral force for the mechanism by Type-IV, in where the capacity may be dominated by the tension compression failure of existing boundary columns, may be calculated by equation (9.41)

$${}_bM_u = \min [\, T_u, N_u \,].l \qquad ....(9.41)$$

where, $T_u = N_1 + (\, \sigma_y . \, A_g)$

$N_u = 0.8 \; f_c \; b \; h + (\sigma_y . A_g) - N_2$

N_1 = axial force by gravitational load at tension side.

N_2 = axial force by gravitational load at compression side.

fc = concrete strength.

b = width of column.

h = depth of column.

l = total length of panel plate, or steel bracing

9.5 Design Techniques for Post-installed Anchors:

Two types of anchors may be applied to joint the new strengthening members/elements to the existing reinforced concrete structure, chemical adhesive anchors and mechanical wedge anchors. The typical shape of them are shown in Fig. 9.13.

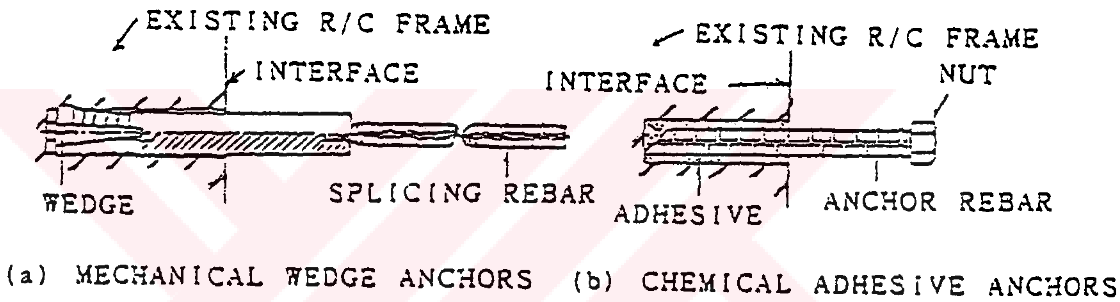


Fig. 9.13 Types of post-installed anchors

9.5.1 Design and Calculation Equations:

Basic anchor strength is defined as the yield strength of post-installed anchor or its bond strength. The design strength should be taken the smallest of the basic anchor strength or the strength of other failure mechanism by a post-installed anchor.

i) <u>Notations related to the equations for calculation of shear and tension strength of post-installed anchor</u>:

As shown in **Fig. 9.14** the following notation will be used in calculation of shear, and tension strength of post-installed anchor . [14]

T_a = tension strength per a post-installed anchor, [N]

T_{a1} = tension strength per a post-installed anchor, that is dominated by its yield strength, [N]

T_{a2} = tension strength per a post-installed anchor, that is dominated by embedded concrete strength, [N]

l = anchor length, [mm]

l_e = effective anchorage length, [mm]

l_a = entire length of a mechanical wedge anchor, [mm]

l_1 = embedded length of mechanical wedge anchor, [mm]

l_2 = projected length of mechanical wedge anchor from interface, [mm]

$l_a = l_1 + l_2$

l_d = total length of steel bolt or rebar which is spliced with post-installed anchor, [mm]

l_n = effective anchorage length of spliced steel bolt or rebar, [mm]

d_a = diameter of post-installed anchor, [mm], or nominal diameter of mechanical wedge anchor, [mm]

d_b = diameter of hole drilled, [mm]

D_a = diameter of wedge, [mm]

A_c = effective area of concrete corn projected by a post-installed anchor, [mm^2] ,as show in Fig. 9.15

A_e = minimum cross-sectional area of mechanical wedge anchor, [mm^2]

A_o = minimum cross-sectional area of spliced bolt or rebar, [mm^2]

${}_aA_e$ = cross-sectional area of post-installed anchor at interface, [mm^2]

σ_B = compression strength of existing concrete, [MPa]

F_c = design strength of existing concrete, [MPa]

E_c = Young's modulus of existing concrete, [MPa]

σ_y = specified yield of rebar, [MPa]

${}_m\sigma_y$ = yield strength of mechanical wedge anchor, [MPa]

Q_a = shear capacity per a post-installed anchor, [N]

Q_{a1} = shear capacity per a post-installed anchor that is dominated by yield strength of steel, [N]

Q_{a2} = shear capacity per a post-installed anchor that is dominated by bearing strength of concrete, [N]

Q_{a3} = shear strength of post-installed anchor, [N]

τ_a = bond strength of chemical adhesive anchor, [MPa]

τ_o = basic bond strength of chemical adhesive anchor, [MPa]

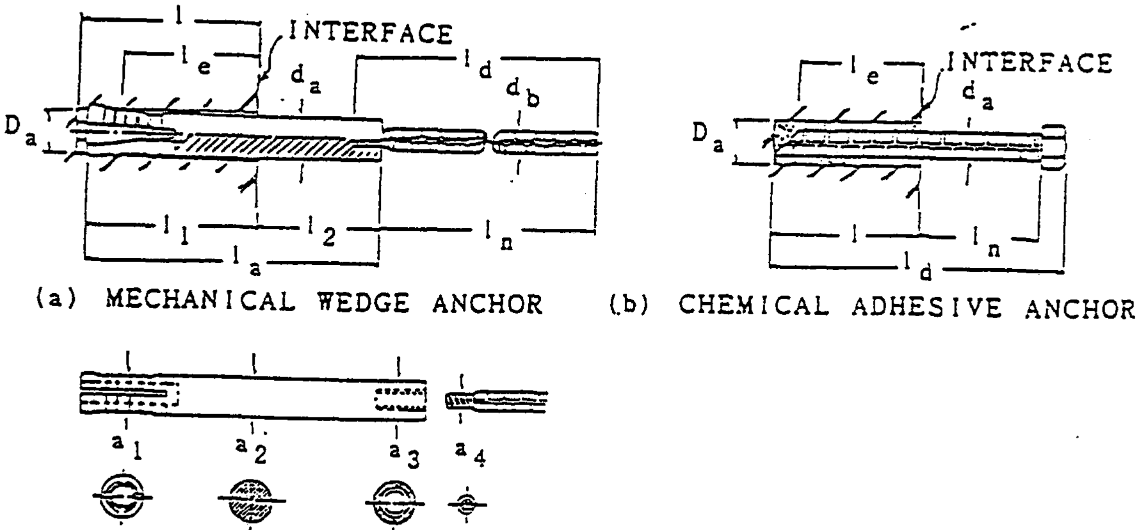


Fig. 9.14 Notation defined for each part of anchors

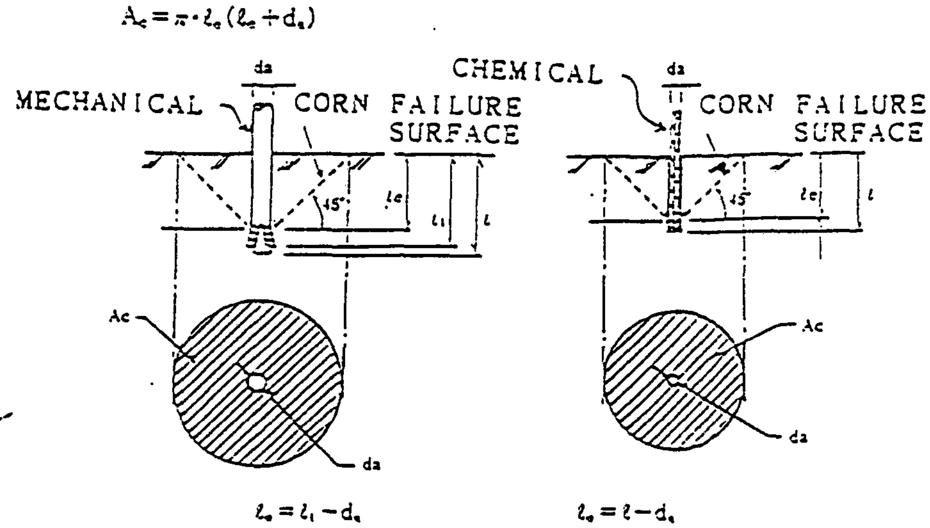


Fig. 9.15 Definition for projection area, A_c

ii) <u>Shear strength of post-installed anchor</u>:

The shear strength of post-installed anchor, Q_a, should be taken the smaller of Q_{a1} (strength dominated by the steel strength of post-installed anchor) or Q_{a2} (strength dominated by the bearing strength of concrete around the post-installed anchor).

$$Q_a = \min [\, Q_{a1} \,,\ Q_{a2} \,] \quad \text{...............(9.42)}$$

where, Q_{a1} and Q_{a2} depend on the effective anchorage length as follows:

(a) for mechanical wedge anchors and $4d_a < l_e < 7d_a$

$$Q_{a1} = 0.7\ {}_{m}\sigma_y \cdot {}_{a}A_a$$

$$Q_{a2} = 0.3\sqrt{E_c \cdot \sigma_B} \cdot {}_{a}A_a$$

But, $Q_a \,/\, {}_{a}A_a$ not exceed 245.25 MPa

(b) for mechanical wedge anchors and $l_e > 7d_a$

$$Q_{a1} = 0.7\ {}_{m}\sigma_y \cdot {}_{s}A_e$$

$$Q_{a2} = 0.4 \sqrt{E_c \cdot \sigma_B} \cdot {}_{s}A_e$$

But, Q_a / ${}_{s}A_e$ shall not exceed 294 MPa

(c) for chemical adhesive anchors and $l_e > 7d_a$

$$Q_{a1} = 0.7\ \sigma_y \cdot {}_{s}A_e$$

$$Q_{a2} = 0.4 \sqrt{E_c \cdot \sigma_B} \cdot {}_{s}A_e$$

But, Q_a / ${}_{s}A_e$ shall not exceed 2940 MPa.

iii) <u>Tension strength of post-installed anchor</u>:

The tension strength of post-installed anchor should be taken the smaller of T_{a1} (strength dominated by the yield strength of steel). T_{a2} (strength dominated by the corn failure strength of concrete) or T_{a3} (strength dominated by the bond failure strength of an embedded post-installed anchor).

(a) for mechanical wedge anchors

$$T_{a1} = \min [\ {}_{m}\sigma_y \cdot A_e \ ,\ \sigma_y \cdot A_o\ ]$$
$$T_{a2} = 0.75\ \sigma_B \cdot A_c$$

(b) for chemical adhesive anchors

$$T_{a1} = \sigma_y \cdot A_o$$
$$T_{a2} = 0.75\ \sigma_B \cdot A_c$$
$$T_{a3} = \tau_a \cdot \pi \cdot d_a \cdot l_e$$

where $\tau_a = 100\ \sigma_B /210$

9.5.2 Requirements and Provisionals:

According to the Japanese techniques the following requirements and provisionals should be followed: [14]

(1) Common Items

i) The chemical adhesive anchor shall be installed to where the anchors are expected to resist the tension corresponding to the yield strength of shear reinforcement in a wall. The embedment length of chemical adhesive anchors shall be more than ten times of diameter of the anchor.

ii) The diameter, pitch or arrangement of post-installed anchors should comply with; (see Fig. 9.16)

diameter : more than 13 mm , but less than 22 mm.

pitch : more than $7.5d_a$, but less than 300 mm.

gauge : more than $5.5d_a$ for double gauges, more than $4d_a$ for stagger arrangement

end margin : more than $5d_a$

side margin : more than $2.5d_a$, post-installed anchor should not be arranged in the cover concrete.

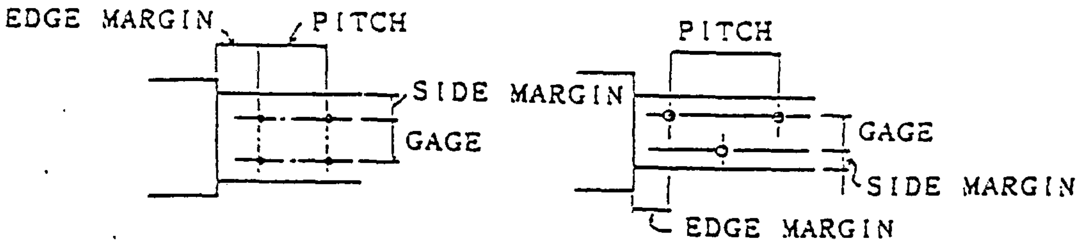
iii) The necessary reinforcement should be arranged to prevent the splitting failure of concrete at the joint region with post-installed anchors.

iv) The post-installed anchors, in general, should be arranged to both the columns and beams which surround the in-filled shear wall.

(2) <u>Mechanical wedge anchors</u>:
The splicing rebars with the post-installed anchors should be the deformed rebars, and the anchorage length into the in-filled shear wall should be more than $30d_a$. The anchorage length may be more than $20d_a$ if the rebar has a hock or a nut.

The effective embedment length of a mechanical wedge anchor, l_e, should be more than $4d_a$, and the projecting length, l_2, should be more than $5d_a$.

(3) <u>Chemical adhesive anchors</u>:
The post-installed anchors, in general, should be the deformed rebars with a nut or a hock, and the anchorage length into the in-filled shear wall should be more than $20d_a$. The embedment length, l_e, should be more than $7d_a$.



(a) DOUBLE ARRANGEMENT (b) STAGGERED ARRANGEMENT

Fig. 9.16 Pitch or gauge for post-installed anchors

CONCLUSION

The following conclusions can be drawn from the present study:

1- Various techniques for increasing the strength and ductility of reinforced concrete members and buildings has been briefly reviewed.
2- From the literature, it is clear that there is no unanimity of opinion among engineers as to the proper methodology for strengthening an existing concrete building.
3- Repair and/or strengthening of damaged and/or weak building is much more complicated problem than it seems, its rational solution is not yet developed, but what found from the literature is a first presentation of some aspects of the problem and some provisional practical suggestions.
4- The investigation, evaluation and determination of whether strengthening is required or not and how to strengthen, is still as much an art as it is a science.
5- The evaluation of damage from any action and identifying it by direct observation, instrumental measurements and numerical analysis, provides basic information for the formation of remedial solutions.
6- Repairing and strengthening techniques for restoring or improving the properties of reinforced concrete components damaged by earthquake-type loading were studied, including repair by injection of epoxy resin, replacing damaged concrete, jacketing members and addition of new structural elements.

7- Repairing by epoxy was found by many researchers effective with high quality control, and must be protected against temperature variations especially against fire, because its strength properties is reduced extensively during and after fire exposure.
8- When repairing and strengthening building damaged by earthquake, it is very important to element the cause of the failures, when possible.
9- The strengthening solutions must be compatible with functional usage of the building which will strongly influence the solution selected.

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CURRICULUM VITAE

The author was born in Tobruk Libya in 1948. He received his B.s degree from University of Triploli, Faculty of Engineering, Civil Engineering Department in June 1974. He worked as a Project Engineer for Housing Department of Tobruk Municipality for three years. He also worked as a Minster of Housing in Tobruk Region from 1976 through 1985. He has consulted in the area of design, construction of houses, schools and general buildings.

From 1985 until 1990, he worked as technical consultant for Municipality of El-Batnanin Tobruk for design, construction, and repairing of old buildings and utilities of Tobruk region. He has the opportunity to be member of the committee which prepare design , construction and preparing specifications of buildings including dowelling houses, schools, general public buildings for the general development plane in Libya since 1976 to 1985.