8-6. Domestic training materials and syllabus

1st Domestic Training

Venue: PWD Seminar Room

Duration: Feb 11, 2013 to Feb 19, 2013

Course Title: **Short Training Course on Seismic Assessment, Retrofit Design and Construction of RC Buildings.**

Course Duration: 11/02/13 to 19/02/13

Program Schedule:

Date	Time		Title of Lecture	Resource Person
11-02-13 (Monday)	2:30 – 3:15		Inauguration of the course	
	3:15 – 3:30		Tea Break	
	3:30 – 5:30	A1 & A2	Basic concept on seismic evaluation of RC buildings	Md. Rafiqul Islam
12-02-13 (Tuesday)	2:30 – 3:25	A3	Overview of seismic evaluation according to Japanese Standard	Ahmed Abdullah Noor
	3:25 – 4:20	A4	Screening procedure with example (1^{st} level)	Md. Emdadul Huq
	4:20 – 4:35		Tea Break	
	4:35 – 5:30	A5	Screening procedure with example (2^{nd} level) [cont.]	Md. Emdadul Huq
13-02-13 (Wednesday)	2:30 – 3:25	A6	Screening procedure with example (2^{nd} level)	Md. Emdadul Huq
	3:25 – 4:20	R1	Concept on Retrofitting Design	Anup Kumar Halder
	4:20 – 4:35		Tea Break	
	4:35 – 5:30	R2	Retrofitting design methods (cont.)	Md. Mominur Rahman
14-02-13 (Thursday)	2:30 – 3:25	R3	Retrofitting design methods	Md. Mominur Rahman
	3:25 – 3:40		Tea Break	
	3:40 – 5:30	R4 & R5	Retrofitting works procedure with site visit	Md. Sohel Rahman
17-02-13 (Sunday)	2:30 – 3:25	R6	Retrofitting design example of a real structure (cont.)	Anup Kumar Halder
	3:25 – 4:20	R7	Retrofitting design example of a real structure	Anup Kumar Halder
	4:20 – 4:35		Tea Break	
	4:35 – 5:30	R8	Pushover Analysis in retrofitting design	Moniruzzaman Moni
18-02-13 (Monday)	2:30 – 3:25	N1	Seismic design concept for new buildings	Md. Mominur Rahman
	3:25 – 4:20	N2	Code provision for seismic analysis with example (cont.)	Md. Rafiqul Islam
	4:20 – 4:35		Tea Break	
	4:35 – 5:30	N3	Code provision for seismic analysis with example	Md. Rafiqul Islam
19-02-13 (Tuesday)	2:30 – 4:20	N4 & N5	Code provision for seismic design with example	Md. Rafiqul Islam
	4:20 – 4:35		Tea Break	
	4:35 – 5:30	N6	Effect of infill wall in seismic analysis	Md. Jahidul Islam Khan
20-02-13 (Wednesday)			Closing ceremony	

1.	d. Ahsan abib, PEng Executive Engineer PWD Design Division 2	2.	ohammad Shamim Akhter Executive Engineer PWD Design Division 6
3.	d. hairul Islam Executive Engineer Staff Officer to Chief Engineer PWD	4.	. . ostafa asan Executive Engineer PWD Audit Division
5.	d. Shakhawat ossain Executive Engineer , PWD	6.	Dr. ohammad Sharfuddin Sub Divisional Engineer PWD Design Division 6
7.	Abdullah ohammod ubair Sub Divisional Engineer PWD Design Division 5	8.	D. Shafiul Islam Assistant Engineer PWD Design Circle 1
9.	d. aruful o ue Sub Divisional Engineer PWD Design Division 2	10.	Nur E awonine Assistant Engineer PWD Design Division 1
11.	A.S. Shahriar Jahan Assistant Engineer PWD Design Division 1	12.	Dewan ehidi assan Assistant Engineer PWD Design Division 1
13.	Sk. Toufi ur Rahman Assistant Engineer PWD Design Division 3	14.	D. Shamsul Islam Assistant Engineer PWD Design Division 4
15.	Lintu a i Assistant Engineer PWD Design Division 6	16.	A. . . Sajadur Rahman Assistant Engineer PWD Design Division 5
17.	Afro a Begum Executive Engineer Education Engineering Department	18.	Shah Naimul uader Executive Engineer (Design) Education Engineering Department
19.	d. Na mul asan Assistant Engineer B/R Army ead uarter, E in C's Branch, Works Directorate (.E.S)	20.	d. Arifujjaman Research Engineer BRI
21.	Dr. Ruhul Amin Senior Engineer DP Consultant	22.	Engr. d. oynal Abedin Additional Chief Engineer Concord Architects and Engineers Ltd.
23.	d. ahedul Islam Executive Engineer L ED	24.	. . Ehsan ameel Project anager RAJ
25.	ahna Tabassum Engineer Axis Design Consultant Ltd.	26.	Engr. B. . Nural Absar Assistant Engineer BCL Associates Ltd.
27.	a i d. Jahangir ossain PEng Executive Engineer National ousing Authority	28.	uhammad aniru aman Deputy Project Director CSSED Project Election Commission secretariat
29.	d. aheb ossain Assistant Engineer Dhaka South City Corporation		

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

Basics of Seismic Vulnerability Assessment

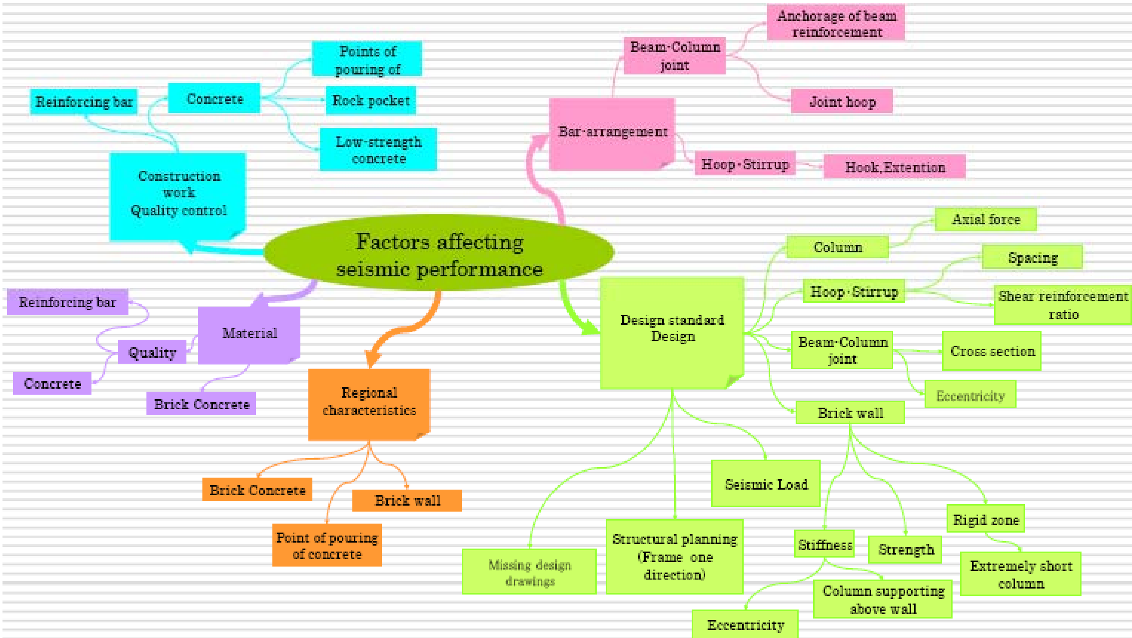
Md. Rafiqul Islam
Executive Engineer
PWD Design Division – 3
&
Team Leader
Working Team – 2
CNCRP Project

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

Is the Building Safe to Earthquake?

- What is the seismic intensity?
- What is the lateral load resisting system?
- What is the performance objective?
- Age of the building?
- Subsoil condition?
- Irregularity of the building?
- What is the evaluation standard?

Factors Affecting Seismic Performance



Reference: Presentation 'Issues of Seismic Performance' by Yosuke Nakajima, JICA Expert Team, 2012

Selection in Performance Based Design

Operational *Immediate Occupancy* *Life Safety* *Collapse Prevention*

Operational – negligible impact on building and it is fully operable

Immediate Occupancy – building is safe to occupy and retain its pre-earthquake strength and stiffness

Life Safety – building is safe during event but possibly not afterward

Collapse Prevention – building is on verge of collapse, probable total loss

Performance Level for Evaluation

Performance level checked for both structural and non-structural components:

1. Life Safety (LS) Performance Level:
   - Partial or total structural collapse does not occur
   - Damage to non-structural components is non-life-threatening

2. Immediate Occupancy (IO) Performance Level:
   - Vertical and lateral force resisting system retain nearly pre-earthquake strength
   - The damage is repairable while the building is occupied

Geologic site hazard and foundation hazard are also assessed.

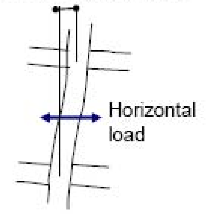
Methods of Seismic Vulnerability Assessment

1. Rapid Visual Screening (FEMA 154) [Pre evaluation sage]
2. Seismic Evaluation of Existing Building (ASCE/SEI 31-03)
   - Tier 1 – Screening phase
   - Tier 2 – Evaluation phase
   - Tier 3 – Detailed evaluation phase
3. Seismic Evaluation of Existing Reinforced Concrete Structure, 2001 [Japanese standard]
4. Euro Code 8: Part 1-4
5. Document by New Zealand Society for Earthquake Engineering
6. Report by Structural Engineering Research Centre of India

Three levels of screening in Japanese Standard

1. First level screening
   - Beam is extremely rigid and only vertical member will deform
   - Vertical members are classified into three categories
   - Concrete strength and sectional area of vertical member are required for calculation; reinforcement details is not required
2. Second level screening
   - Beam is extremely rigid and only vertical member will deform
   - Vertical members are classified into five categories
   - Reinforcement details of vertical member is required for calculation
3. Third level screening
   - Beam is flexible and hinge may form either in beam or column
   - Vertical and horizontal members are classified into eight categories
   - Calculation process is very rigorous

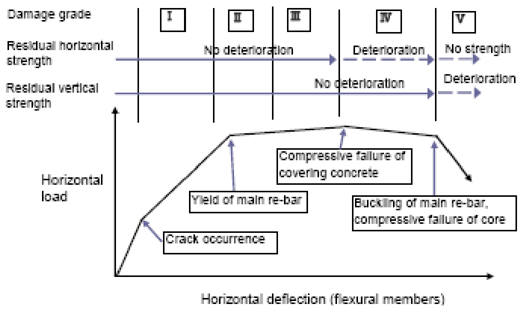
Horizontal Load-Deflection Curve of RC Columns in Japan

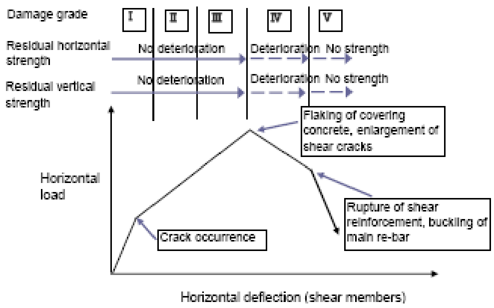


Damage grade Ⅲ

Damage grade Ⅳ

Damage grade Ⅴ

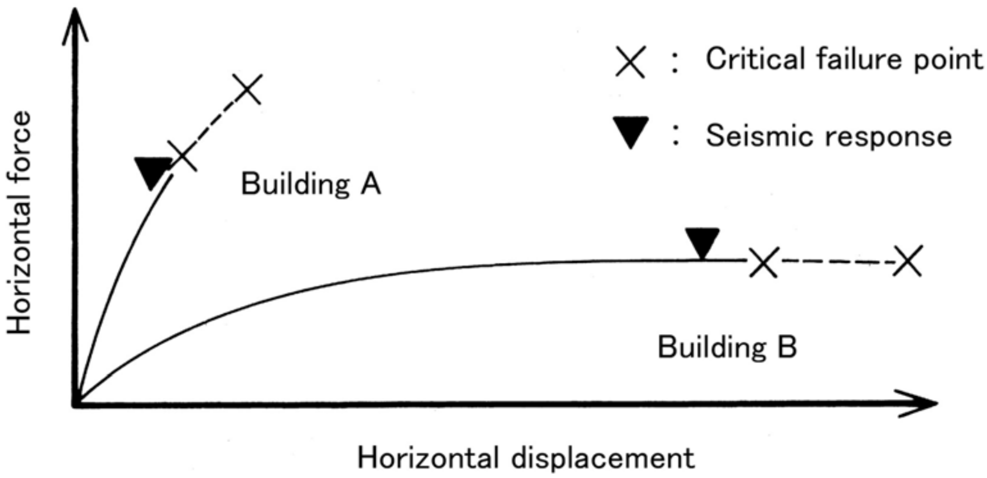




Reference: Lecture 'Seismic damages and performance of building' by Akira Inoue , JICA Expert Team, 07/06/2011

[Source: "Standard of Judgment of Damage Grade and Guidelines of Recovery Engineering for Damaged Buildings, 2001", The Japan Building Disaster Prevention Association (written in Japanese)]

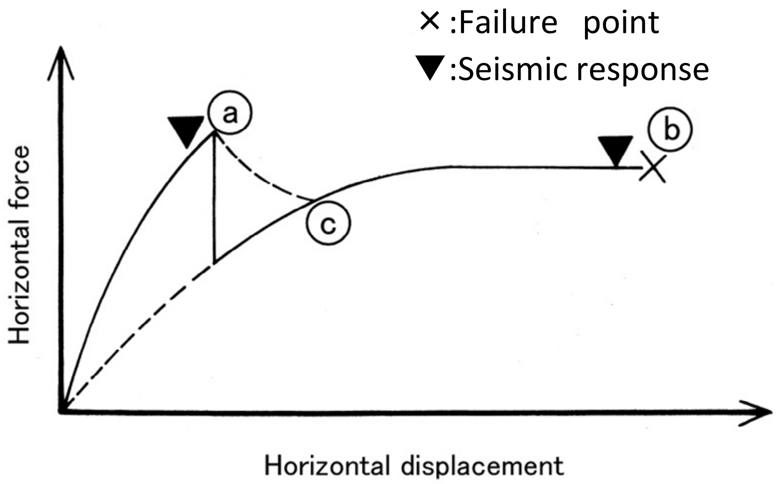
Different Types of Structure



Building 'A' resist earthquake by Strength

Building 'B' resist earthquake by Ductility

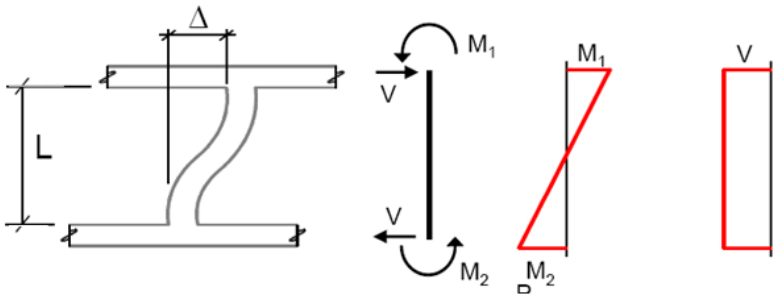
Structure Configuring Members with Different Ductility



Brittle failure of non-ductile member at 'a'
Sudden drop of stiffness from 'a' to 'c'
Performance of ductile member up to 'b'

What is Strength?

A) Flexural strength (Q_{mu}) from moment capacity of the structural member



$$Q_{mu} = \frac{M_1 + M_2}{L}$$

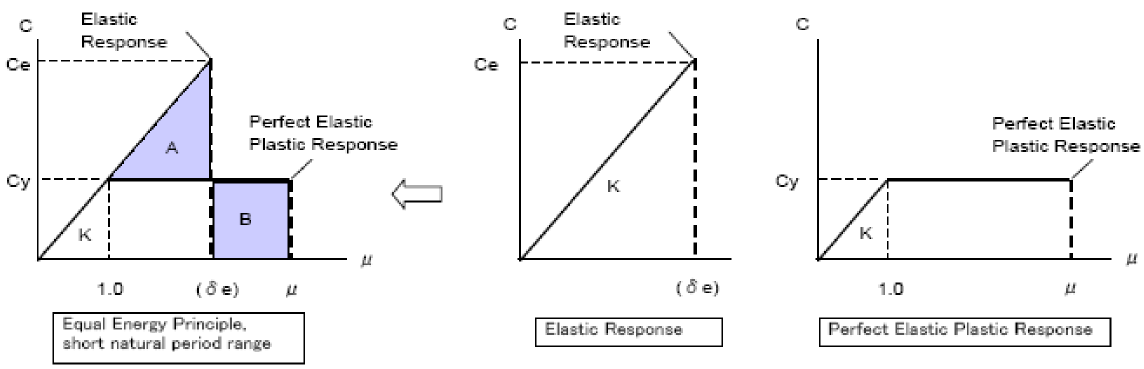
B) Shear strength (Q_{su}) from shear reinforcement of the structural member

Strength is minimum of Q_{mu} and Q_{su}

What is Ductility?

Ductility is the capacity of building material, systems or structure to absorb energy by deforming into inelastic range.

Equal Energy Principle: Ideal Non-linear Earthquake Response
In a relatively short range period building



F = Ce/ Cy = $\sqrt{2\mu - 1}$ (Newmark's formula)
Area A = Area B
C: Shear force coefficient
μ: Ductility factor

Ref: 'Seismic Design, Evaluation and Retrofitting of Building ' by Akira Inoue, JICA Expert Team

Japanese Method of Seismic Vulnerability Assessment

Seismic index of structure $I_s = E_0 \times S_D \times T$

E_0 = Basic seismic index of structure = C x F

C = Strength index $= \frac{Q}{W}$

Q = Shear strength

W = Weight on vertical member

F = Ductility index (it is function of ductility factor)

S_D = Irregularity index

T = Time index

Seismic Demand Index

Seismic demand index, $I_{so} = E_s \times Z \times G \times U$

E_s = Basic seismic demand index (depends on level of screening)

Z = Zone index (factor for seismic intensity of the site)

G = Ductility index (factor consider sub-soil condition)

U = Usage index (factor for occupancy type)

Judgment on Seismic Safety

A safe structure shall satisfy both the following checking:

A) $I_s \geq I_{so}$

I_s = Seismic index of structure

I_{so} = Seismic demand index

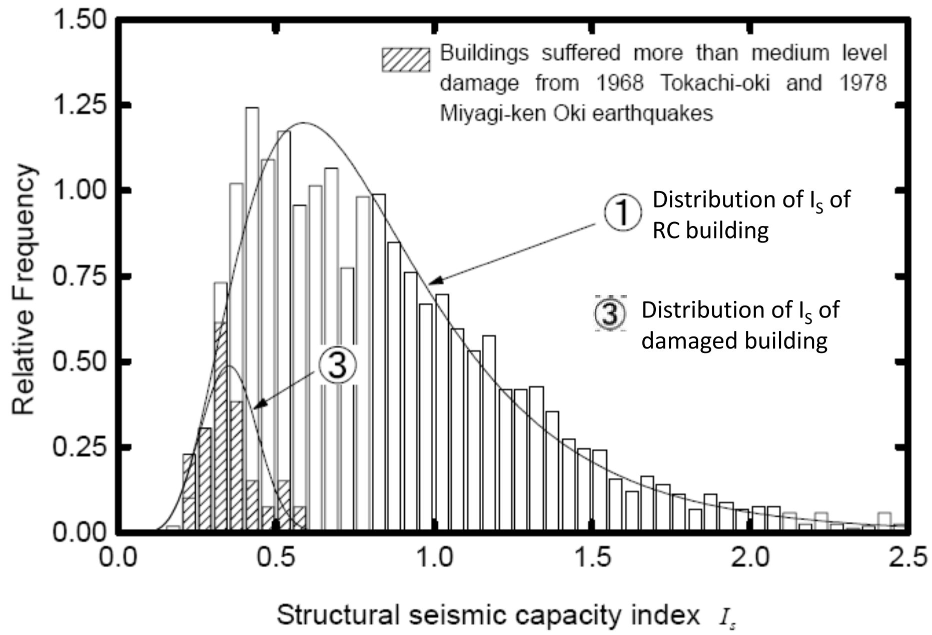
B) $C_{TU} \geq 0.3(?) \times Z \times G \times U$

C_{TU} = Cumulative strength at ultimate deformation of structure

Judgment to be applied

- Each story
- Each principal horizontal direction of a building

Study on Earthquake Damaged Buildings



Ref: Nakano, Yoshiaki and Tsuneo Okada "Reliability analysis on seismic capacity of existing reinforced concrete buildings in Japan" Journal, Transaction of Architecture Institute of Japan, No. 406, 37 – 43 (1988)

I_{SO} for Bangladeshi Buildings

Base shear, $V = \frac{ZIC}{R} W$

Z = Zone coefficient

I = Importance factor

C = Numerical coefficient for sub-soil and structural period (maximum value is 2.75)

Rearranging

$$\frac{V}{W} \times R = Z \times I \times C$$

i.e Strength index × ductility index = Z × I × C

For Dhaka Z × I × C = 0.15 × 1 × 2.75 = 0.413 (?)

Reference: Presentation 'proposed seismic demand index of structures, Iso, for existing RC buildings in Bangladesh' by Akira Inoue , JICA Expert Team, 2012

How to Calculate Strength Index (C)

Strength index, C $= \frac{Q_u}{\Sigma W}$

where ,

Q_u =Ultimate lateral load carrying capacity of vertical member

ΣW = Weight of the building supported by the story concerned

Strength index shall be modified for each story by

Story shear modification factor $= \frac{n+1}{n+i}$

n = Number of stories of a building

i = Number of story is being evaluated

How to Calculate Ductility Index (F)

Various types of deflection angle (R_{max}, R_y, R_{su}, R_{mu}, R_{mp}) of column is calculated based on:

1) Column size
2) Clear height of column
3) Axial force ratio
4) Shear force ratio
5) Tensile reinforcement ratio
6) Spacing of shear reinforcement
7) Margin against shear failure
8) etc.

How to Calculate Ductility Index (F)

A) Ductility index for shear column:

$$F = 1.0 + 0.27\frac{R_{su} - R_{250}}{R_y - R_{250}}$$

B) Ductility index for flexural column:

(i) In case $R_{mu} < R_y$

$$F = 1.0 + 0.27\frac{R_{mu} - R_{250}}{R_y - R_{250}}$$

(ii) In case $R_{mu} \geq R_y$

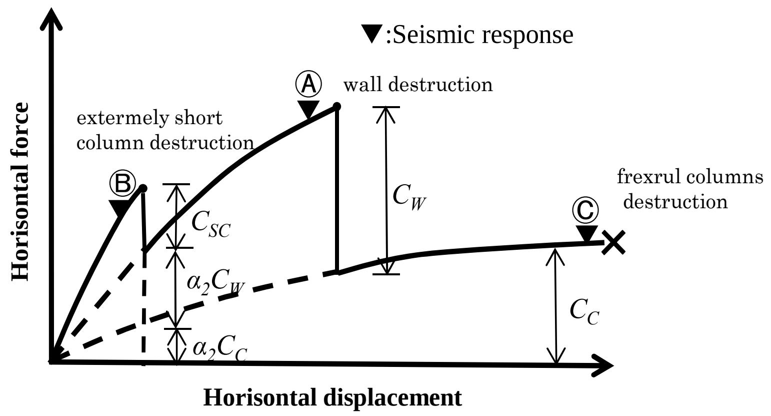
$$F = \frac{\sqrt{2\,R_{mu}/R_y - 1}}{0.75 \cdot \left(1 + 0.05\,R_{mu}/R_y\right)} \leq 3.2$$

Strength Dominant Basic Seismic Index (E_0)

$$E_0 = \frac{n+1}{n+i}\left(C_i + \sum_j \alpha_j C_j\right) \cdot F_1$$

where:

αj= Effective strength factor



Ductility Dominant Basic Seismic Index (E_0)

$$E_0 = \frac{n+1}{n+i}\sqrt{{E_1}^2 + {E_2}^2 + {E_3}^2}$$

where:

$E_1 = C_1 \times F_1$
$E_2 = C_2 \times F_2$
$E_3 = C_3 \times F_3$

C_1 = The strength index *C* of the first group (with small *F* index).
C_2 = The strength index *C* of the second group (with medium *F* index).
C_3 = The strength index *C* of the third group (with large *F* index).
F_1 = The ductility index *F* of the first group.
F_2 = The ductility index *F* of the second group.
F_3 = The ductility index *F* of the third group.

Irregularity Index (S_D)

Irregularity Index covers:

1. Regularity
2. Aspect ratio of plan
3. Expansion joint
4. Well-style area
5. Underground floor
6. Story height uniformity
7. Soft story
8. Eccentricity
9. Stiffness/mass ratio

Irregularity Index (S_D) $\leq$ 1.0

Time Index (T)

Time Index evaluates:

1. Deflection of beam and column
2. Cracking in walls
3. Fire experience
4. Occupied by chemical
5. Age of building
6. Finishing condition

Time Index (T) $\leq$ 1.0

Second-Class Prime Element

A column is a Second-Class Prime Element if

1. It fails in brittle manner
2. Its sustaining axial load can not be redistributed or not be sustained by surrounding members in the structure
3. Lateral force resisting capacity of the structure is still enough

It is necessary to check

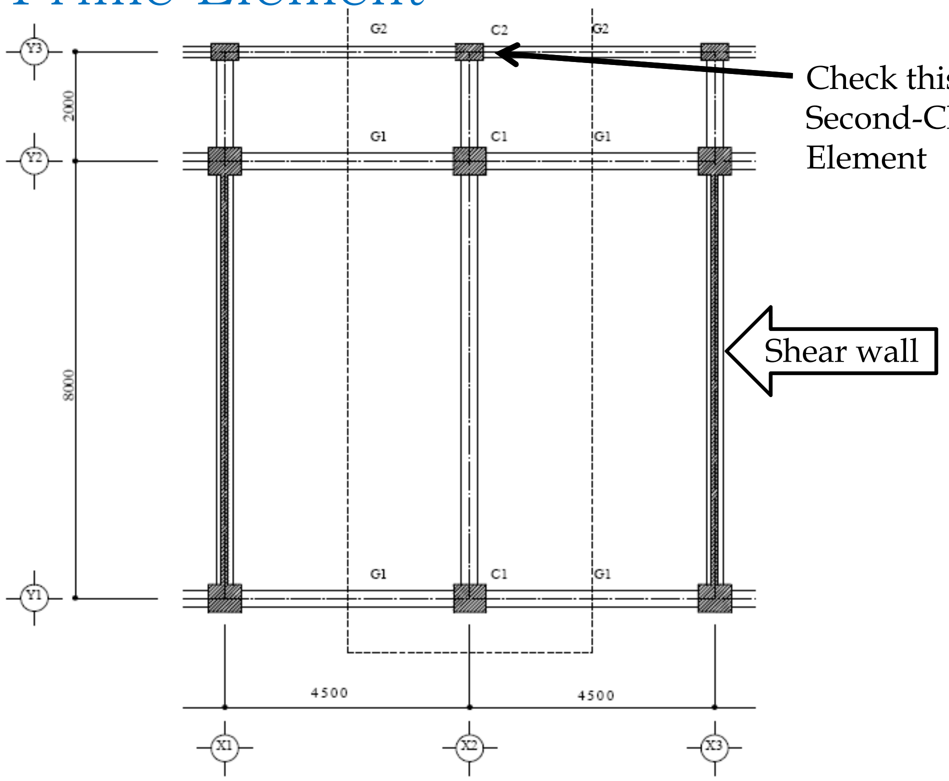
- Shear column
- Extremely short column
- Column supporting the wall above

Residual Axial Load Capacity, $\eta = N/A_cF_c$

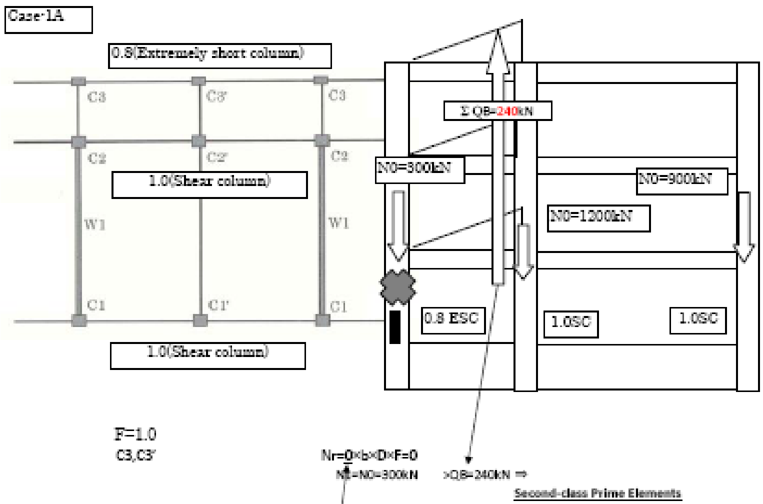
Column	p_w (%)	F=1.0	F=1.27	F=2	F=3
Extremely short column*3	$0.4 < p_w$ *1	0.4	0.3	0.1	0
	$0.2 \leq p_w \leq 0.4$ *2	0.3[0.4]	0.1	0	0
	$p_w < 0.2$	0[0.4]	0	0	0
Shear column	$0.4 < p_w$ *1	0.6	0.4	0.2	0
	$0.2 \leq p_w \leq 0.4$ *2	0.5	0.3[0.4]	0.1	0
	$p_w < 0.2$	0.4	0[0.4]	0	0
Flexural column	$0.4 < p_w$ *1	0.6	0.6	0.5	0.4
	$0.2 \leq p_w \leq 0.4$ *2	0.5	0.5	0.3[0.4]	0.2[0.3]
	$p_w < 0.2$	0.4	0.4	0[0.3]	0[0.2]

*1: In case that spacing is not larger than 100mm, $p_w > 0.4\%$, and sub ties are provided at the same spacing as that of main ties. In case where p_w is different in each direction, the smaller p_w can be used.
*2: In case that spacing is not larger than 100mm.
*3: The flexural column of $h_0/D \leq 2$ and $F < 1.27$ is included.

Example of Second-Class Prime Element



Example of Second-Class Prime Element



Use F=0.8

Table TN.3-1 Residual axial load capacity N_r and axial load capacity N_R
$(\eta_r = N_r / A_C F_C \mid \eta_R = N_R / A_C F_C)$
(quoted from Table 3.2.1-1 in the commentary of 3.2.1 of the Standard of 2001 Japanese version)

Column	p_w (%)	F=1.0	F=1.27	F=2	F=3
Extremely short column*3	$0.4 < p_w$ *1	0.4	0.3	0.1	0
	$0.2 \le p_w \le 0.4$ *2	0.3[0.4]	0.1	0	0
	$p_w < 0.2$	0[0.4]	0	0	0
Shear column	$0.4 < p_w$ *1	0.6	0.4	0.2	0
	$0.2 \le p_w \le 0.4$ *2	0.5	0.3[0.4]	0.1	0
	$p_w < 0.2$	0.4	0[0.4]	0	0
Flexural column	$0.4 < p_w$ *1	0.6	0.6	0.5	0.4
	$0.2 \le p_w \le 0.4$ *2	0.5	0.5	0.3[0.4]	0.2[0.3]
	$p_w < 0.2$	0.4	0.4	0[0.3]	0[0.2]

Reference: Lecture material by Yosuke Nakajima, JICA Expert Team

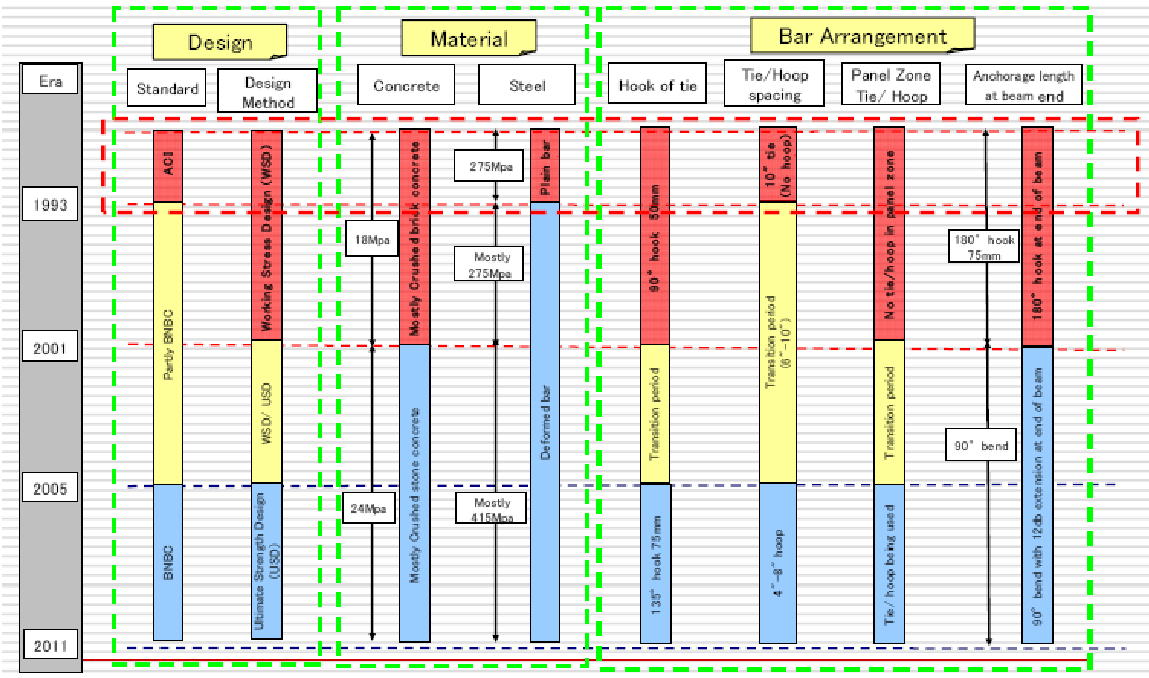
Building Inspection

Inspection Types	Inspection Objectives	Inspection Items
Preliminary inspection	To determine the applicability of the evaluation standard	Summery of the structure and building condition
Inspection without design drawings	To inspect various structural elements by conducting the actual measurement	The dimensions of building frames and reinforcing bars, arrangement of bars, etc
Detailed inspection	▪To calculate Time index and irregularity index ▪To inspect the necessity of refurbishment of aged deterioration ▪To determine the present strength related data to enhance the accuracy of evaluation procedure	Differences from original design drawings, structural cracks, deformations. Inspect material strength, concrete neutralization depth, reinforcing bar strength etc.

Benchmark for Buildings of USA

	Model Building Seismic Design Provisions							
Building Type[1, 2]	NBC[ls]	SBC[ls]	UBC[ls]	IBC[ls]	NEHRP [ls]	FEMA 178[ls]	FEMA 310[ls, io]	CBC[io]
Wood Frame, Wood Shear Panels (Type W1 & W2)	1993	1994	1976	2000	1985	*	1998	1973
Wood Frame, Wood Shear Panels (Type W1A)	*	*	1997	2000	1997	*	1998	1973
Steel Moment-Resisting Frame (Type S1 & S1A)	*	*	1994[4]	2000	**	*	1998	1995
Steel Braced Frame (Type S2 & S2A)	1993	1994	1988	2000	1991	1992	1998	1973
Light Metal Frame (Type S3)	*	*	*	2000	*	1992	1998	1973
Steel Frame w/ Concrete Shear Walls (Type S4)	1993	1994	1976	2000	1985	1992	1998	1973
Reinforced Concrete Moment-Resisting Frame (Type C1)[3]	1993	1994	1976	2000	1985	*	1998	1973
Reinforced Concrete Shear Walls (Type C2 & C2A)	1993	1994	1976	2000	1985	*	1998	1973
Steel Frame with URM Infill (Type S5, S5A)	*	*	*	2000	*	*	1998	*
Concrete Frame with URM Infill (Type C3 & C3A)	*	*	*	2000	*	*	1998	*
Tilt-up Concrete (Type PC1 & PC1A)	*	*	1997	2000	*	*	1998	*
Precast Concrete Frame (Type PC2 & PC2A)	*	*	*	2000	*	1992	1998	1973
Reinforced Masonry (Type RM1)	*	*	1997	2000	*	*	1998	*
Reinforced Masonry (Type RM2)	1993	1994	1976	2000	1985	*	1998	*
Unreinforced Masonry (Type URM)[5]	*	*	1991[6]	2000	*	1992	*	*
Unreinforced Masonry (Type URMA)	*	*	*	2000	*	*	1998	*

No Benchmark for Buildings in Bangladesh



Reference: 'Issues of Seismic Performance' by Yosuke Nakajima, JICA Expert Team

Difficulties of Seismic Assessment of Bangladeshi Buildings

1. Missing architectural and structural design of existing building.
2. Lack of reliability in construction even if drawing is available.
3. A few or no study about lateral load resisting system of building of our country.
4. Effect of infill masonry wall in frame structure.
5. Performance of mixed type (masonry + RC frame) structure.
6. Reinforcing bars of existing structure are significantly corroded.
7. Etc.

Thank you

CNCRP

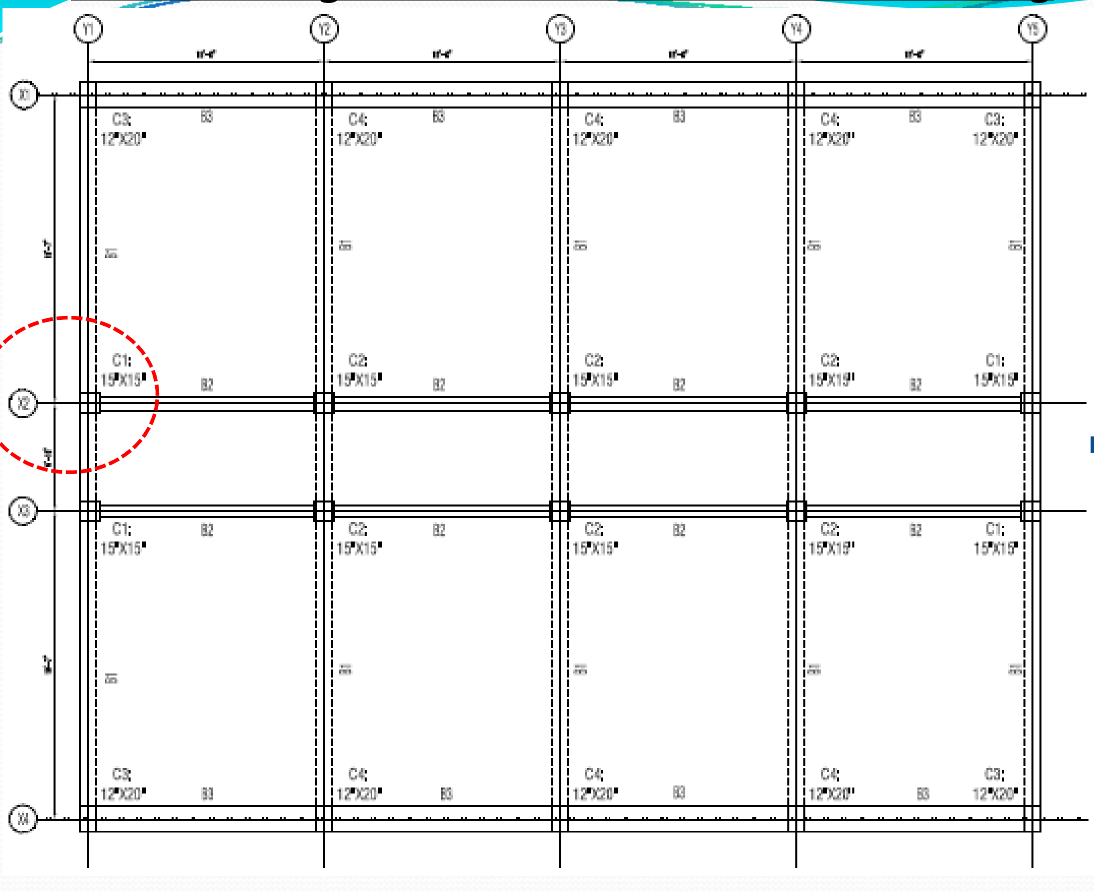
(CAPACITY DEVELOPMENT ON NATURAL DISASTER RESISTANT TECHNIQUES OF CONSTRUCTION AND RETROFITTING FOR PUBLIC BUILDINGS)

Technical co-operation project between –PWD & JICA

PRESENTATION ON
Screening Procedure with example (1^{st} & 2^{nd} Level)

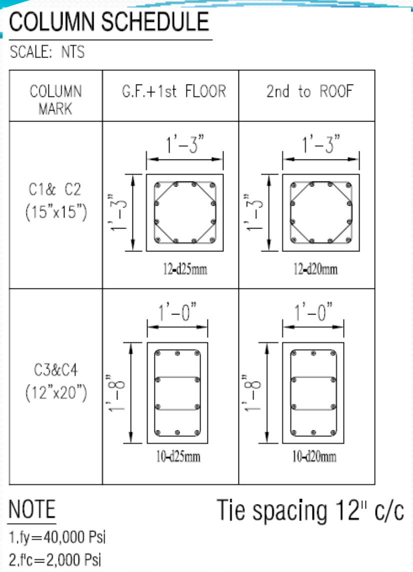
Md. Emdadul Huq
Member of Working Team-2

PLAN : Bangladesh Secretariat Clinic Building

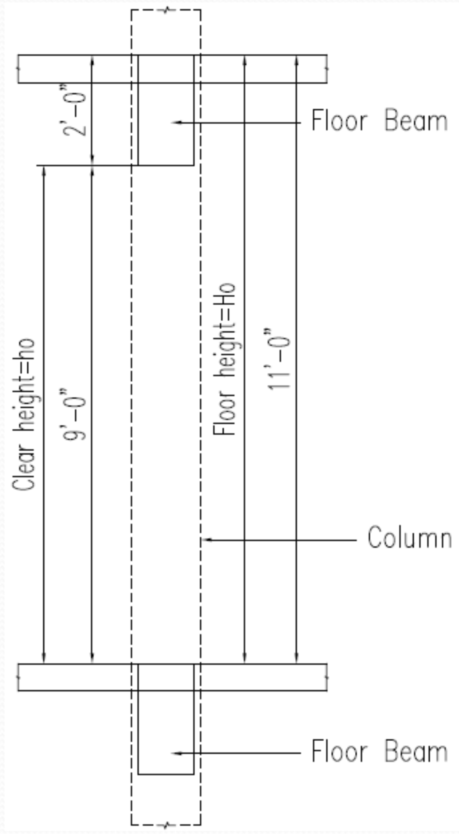


Studied Direction

Structural Drawing



BEAM MARK	SIZE
B1	12"X24"
B2	12"X22"
B3	12"X22"



About The Building

1. Name: Bangladesh Secretariat Clinic(Hospital Building)
2. Design period of the building is 1984
3. 5(Five)-Storied framed structured building.
4. Seismic detailing not provided
5. f'c = 2000 Psi = 13.79 N/mm^2
6. fy = 40000 Psi = 275 N/mm^2

Japanese Standard (Contd..)

In the **Japanese standard** three levels of seismic screening procedure.

1) 1st level screening procedure.
2) 2nd level screening procedure.
3) 3rd level screening procedure.

1st level:

-Simplest
(easy to calculation in comparison with other two evaluation procedure)
-More conservative
-Only X-sectional area & Concrete Strength of vertical Member is considered to calculate the strength
-Inelastic deformability is neglected in this level.

Japanese Standard

2nd level:

-More detail than 1st level screening procedure.
-Assuming that the strength of beam is greater than that of column(Weak column & Strong Beam)
-Evaluate ultimate strength & plastic deformation capacity of vertical members based on x-section, bar detail & material strength.

3rd level:

-Building characteristics are examined in greater detail than in the 2nd level screening procedure
-3rd level is more reliable than 2nd level screening procedure where weak beam in structure

Basic Concept of Seismic evaluation

Seismic Index of Structure (Is) = $E_o \times S_d \times T$

Where

E_o = Basic Seismic Index of Structure

= C X F

C= Strength Index

F=Ductility Index

S_d = Irregularity Index

T = Time Index

UNDERSTANDING FAILURE MODE

(STRUCTURE TYPE AND FAILURE MODE)

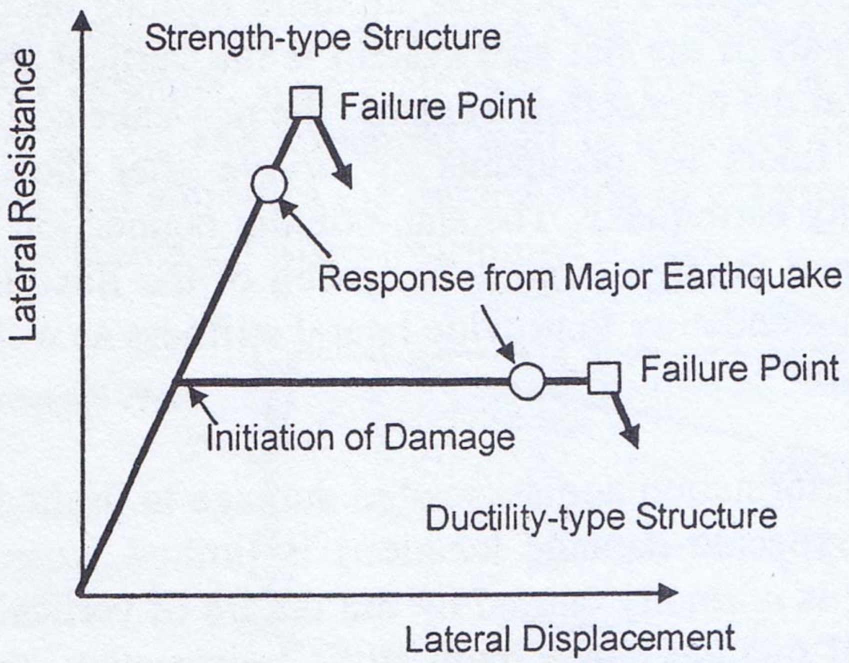
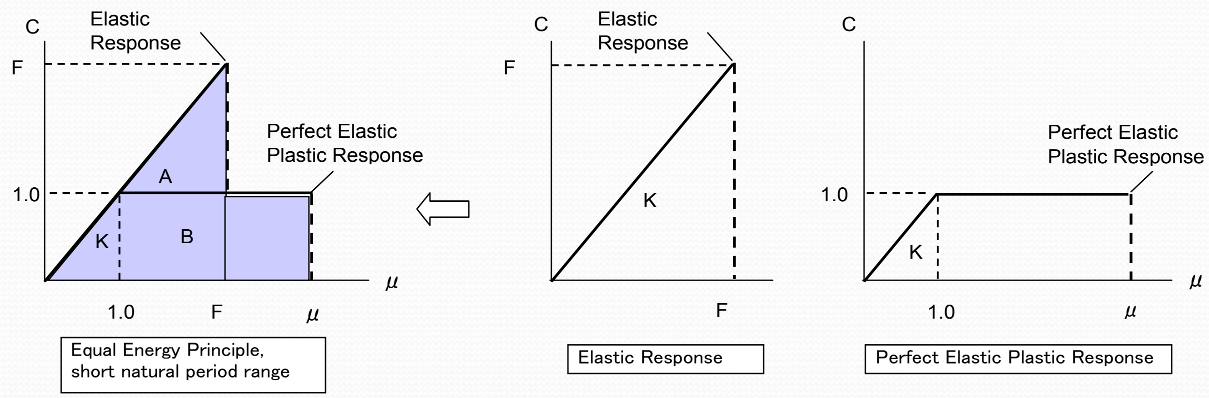


Fig. : Strength and Ductility of a Structure

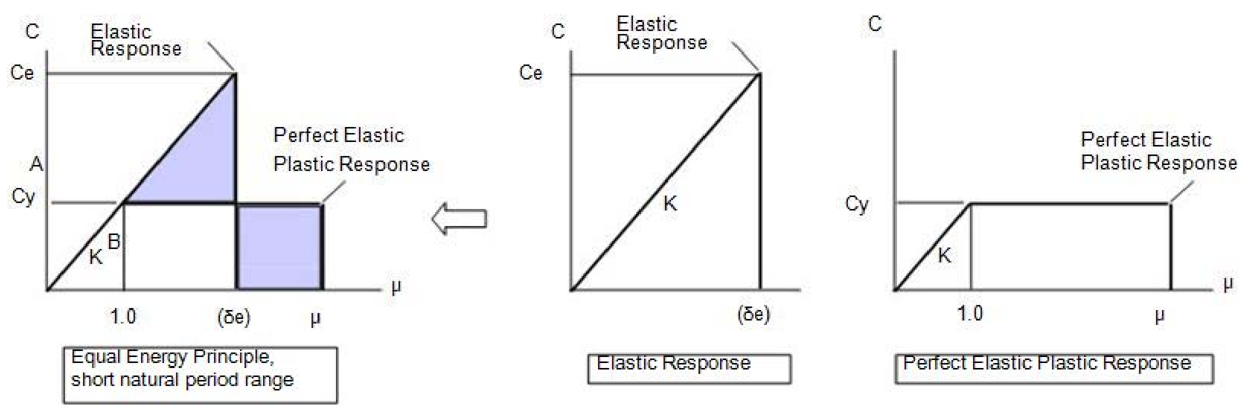
Basic Concept of Eo (Contd..)

"C x F" shows basic seismic performance of a structure. "C" is strength index, which is horizontal strength divided by building weight. "F" is ductility index. This "F" is developed based on (so called) Newmark's principle, and is related to ductility factor μ as shown below. This Equal Energy Principle for an ideal non-linear earthquake response is accepted practically in case of buildings with relatively short range natural period.



$F = R = \sqrt{2\mu - 1}$ (Newmark's formula)
Area A = Area B
μ: Ductility factor
C: Shear force coefficient

BASIC CONCEPT OF Eo (Contd..)



Cy = Min Base Shear Coefficient Structural System(Elastic Plastic Response)
Ce =Ground Motion Produces Elastic Response Base Shear(Elastic Response)
μ= Ductility(Ultimate Deformation/Yield Deformation)

$$C_e = C_y\sqrt{2\mu - 1} \quad \text{for short period systems}$$

$$C_e = C_y \cdot \mu \quad \text{for long period systems}$$

$$E_0 = C \cdot F$$

BASIC SEISMIC INDEX OF STRUCTURE (Eo) Contd…

Two Types Seismic Index

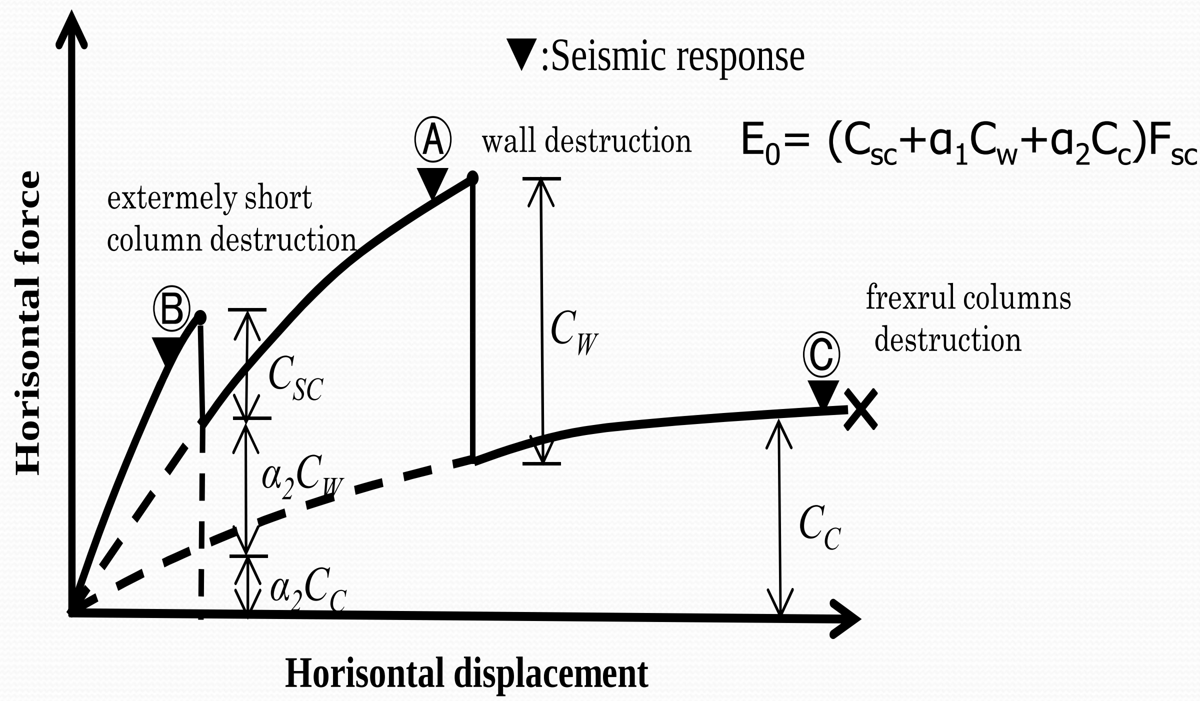
A. Strength-Dominant Structure

$$E_0 = \frac{n+1}{n+i}\left(C_1 + \sum_j \alpha_j C_j\right) \cdot F_1$$

B. Ductility-Dominant Structure

$$E_0 = \frac{n+1}{n+i}\sqrt{E_1^2 + E_2^2 + E_3^2}$$

A) Strength-Dominant Structure (Contd.)



C-F Relation Among Extremely brittle Column, Shear Wall & Flexural Column

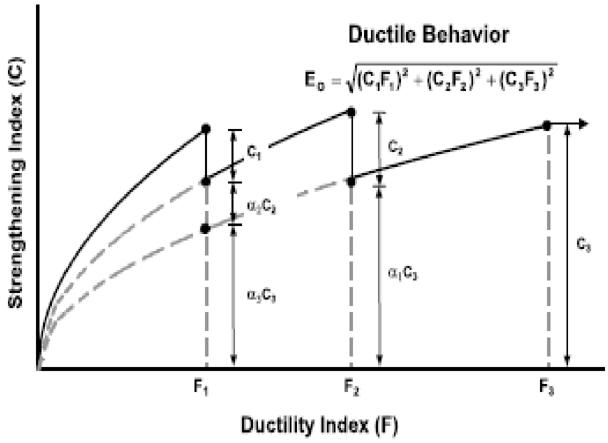
B) Ductility-Dominant Structure (Contd.)

$$E_0 = (n+1)/(n+i)\sqrt{(E_1^2 + E_2^2 + E_3^2)}$$

Where

$$E_1 = C_1\ F_1$$
$$E_2 = C_2\ F_2$$
$$E_3 = C_1\ F_3$$

C_1 = The strength index *C* of the first group (with small *F* index).
C_2 = The strength index *C* of the second group (with medium *F* index).
C_3 = The strength index *C* of the third group (with large *F* index).
F_1 = The ductility index *F* of the first group.
F_2 = The ductility index *F* of the second group.
F_3 = The ductility index *F* of the third group.



Classification of vertical members in the 1st level screening procedure

Vertical member	Definition
Column	Columns having h_o/D larger than 2
Extremely short column	Columns having h_o/D equal to or less than 2
Wall	Walls including those without boundary columns

Ductility index in the 1st level screening

Vertical member	Ductility index *F*
Column ($h_0/D>2$)	1.0
Extremely short column ($h_0/D\leq 2$)	0.8
Wall	1.0

Note: h_o : Column clear height
D : Column depth

Basic Seismic Index of Structure(E_0) For 1st level Screening

$$E_0 = \frac{n+1}{n+i}(C_W + \alpha_1 C_C) \cdot F_W$$

$$E_0 = \frac{n+1}{n+i}(C_{SC} + \alpha_2 C_W + \alpha_3 C_C) \cdot F_{SC}$$

Where:
n = Number of stories of a building.
i = Number of the story for evaluation, where the first story is numbered as1 and the top story as n.
C_W = Strength index of the walls.
C_C = Strength index of the columns.
α_1 = Effective strength factor of the columns at the ultimate deformation of the walls, which may be taken as 0.7. The value should be 1.0 in case of C_w=0.
α_2 = Effective strength factor of the walls at the ultimate deformation of the extremely short columns, which may be taken as 0.7.
α_3= Effective strength factor of the columns at the ultimate deformation of the extremely short columns, which may be taken as 0.5.
F_W = Ductility index of the walls , which may be taken as 1.0.
F_{sc} = Ductility index of the extremely short columns, which may be taken as 0.8.

Calculation of Shear Stress & Ductility Index

ST0RY		AT GRID X1Y1
5	COLUMN	C1
	h0/D	9x12/15=7.2
	CATEGORY	COLUMN
	τ(N/mm²)	0.7
	F	1.0
4	COLUMN	C1
	h0/D	7.2
	CATEGORY	COLUMN
	τ(N/mm²)	0.7
	F	1.0
3	COLUMN	C1
	h0/D	7.2
	CATEGORY	COLUMN
	τ(N/mm²)	0.7
	F	1.0
2	COLUMN	C1
	h0/D	7.2
	CATEGORY	COLUMN
	τ(N/mm²)	0.7
	F	1.0
1	COLUMN	C1
	h0/D	7.2
	CATEGORY	COLUMN
	τ(N/mm²)	0.7
	F	1.0

Calculation of Shear Stress & Ductility Index

STORY		X2&Y1	X2&Y2	X1&Y1	X1&Y2
5	COLUMN	C1	C2	C3	C4
	h0/D	7.2	7.2	9.0	9.0
	CATEGORY	COLUMN	COLUMN	COLUMN	COLUMN
	τ(N/mm^2)	0.7	0.7	0.7	0.7
	F	1.0	1.0	1.0	1.0
4	COLUMN	C1	C2	C3	C4
	h0/D	7.2	7.2	9.0	9.0
	CATEGORY	COLUMN	COLUMN	COLUMN	COLUMN
	τ(N/mm^2)	0.7	0.7	0.7	0.7
	F	1.0	1.0	1.0	1.0
3	COLUMN	C1	C2	C3	C4
	h0/D	7.2	7.2	9.0	9.0
	CATEGORY	COLUMN	COLUMN	COLUMN	COLUMN
	τ(N/mm^2)	0.7	0.7	0.7	0.7
	F	1.0	1.0	1.0	1.0
2	COLUMN	C1	C2	C3	C4
	h0/D	7.2	7.2	9.0	9.0
	CATEGORY	COLUMN	COLUMN	COLUMN	COLUMN
	τ(N/mm^2)	0.7	0.7	0.7	0.7
	F	1.0	1.0	1.0	1.0
1	COLUMN	C1	C2	C3	C4
	h0/D	7.2	7.2	9.0	9.0
	CATEGORY	COLUMN	COLUMN	COLUMN	COLUMN
	τ(N/mm^2)	0.7	0.7	0.7	0.7
	F	1.0	1.0	1.0	1.0

Calculation of column strength(C)

$$C_C = \frac{\tau_C \cdot A_C}{\Sigma W} \cdot \beta_c \qquad \beta_c = \frac{F_c}{20} \qquad F_c \le 20$$

$$C_{SC} = \frac{\tau_{SC} \cdot A_{SC}}{\Sigma W} \cdot \beta_c \qquad \beta_c = \sqrt{\frac{F_c}{20}} \qquad F_c > 20$$

C_C = Strength index of columns.

C_{SC} = Strength index of extremely short columns.

τ_C = Average shear stress at the ultimate state of columns, which may be taken as 1 N/mm^2 or 0.7 N/mm^2 in case h_0/D is larger than 6.

τ_{SC} = Average shear stress at the ultimate state of extremely short columns, which may be taken as 1.5 N/mm^2.

A_{SC} = Total cross-sectional area of extremely short columns (mm^2).

A_C = Total cross-sectional area of columns (mm^2)

F_c = Compressive strength of concrete (N/mm^2)

ΣW = Total weight (dead load plus live load for seismic calculation) supported by the story concerned

Calculation of Area Unit Weight(W)

TYPE OF LOAD	TYPICAL FLOOR	ROOF	Unit
Live Load	0.80	0.30	kN/m^2
Brick Wall	4.50	0.00	kN/m^2
Floor Finish	1.25	2.00	kN/m^2
Slab Weight	3.50	3.50	kN/m^2
SW(Column+Beam)	2.25	2.25	kN/m^2
W	12.3	8.05	kN/m^2

Calculation of Floor Weight

STORY	L(m)	B(m)	A(m^2)	∑W
5	18.6	14.54	270.44	2177
4	18.6	14.54	270.44	5504
3	18.6	14.54	270.44	8830
2	18.6	14.54	270.44	12156
1	18.6	14.54	270.44	15483

Calculation of Strength Index of Column (Cc) (Contd...)

Column ID	Story	β_c	∑W	A_c (mm^2)	τ(N/mm^2)	C_c
C1	5	0.69	2177	140625	0.7	0.031
	4	0.69	5504	140625	0.7	0.012
	3	0.69	8830	140625	0.7	0.008
	2	0.69	12156	140625	0.7	0.006
	1	0.69	15483	140625	0.7	0.004

Calculation of Strength Index of Column(Cc)

Column ID	Story	β_c	$\sum W$	A_c (mm^2)	τ(N/mm^2)	C_c
C1	5	0.69	2177	140625	0.7	0.031
	4	0.69	5504	140625	0.7	0.012
	3	0.69	8830	140625	0.7	0.008
	2	0.69	12156	140625	0.7	0.006
	1	0.69	15483	140625	0.7	0.004
C2	5	0.69	2177	140625	0.7	0.031
	4	0.69	5504	140625	0.7	0.012
	3	0.69	8830	140625	0.7	0.008
	2	0.69	12156	140625	0.7	0.006
	1	0.69	15483	140625	0.7	0.004
C3	5	0.69	2177	150000	0.7	0.033
	4	0.69	5504	150000	0.7	0.013
	3	0.69	8830	150000	0.7	0.008
	2	0.69	12156	150000	0.7	0.006
	1	0.69	15483	150000	0.7	0.005
C4	5	0.69	2177	150000	0.7	0.033
	4	0.69	5504	150000	0.7	0.013
	3	0.69	8830	150000	0.7	0.008
	2	0.69	12156	150000	0.7	0.006
	1	0.69	15483	150000	0.7	0.005

Time index *T* by the first level inspection

[A] Item to be checked	[B] Degree	[C] *T* value (check circle at relevant degree)	[D] Item to be checked for the second level inspection
Deflection	Tilting of a building or obvious uneven settlement is observed	0.7	Structural cracking and deflection
	Landfill site or former rice field	0.9	
	Deflection of beam or column is observed visually	0.9	
	No correspondence to the foregoing	1	
Cracking in walls and columns	Rain leak with rust of reinforcing bar is observed	0.8	Structural cracking and deflection
	Inclined cracking in columns is obviously observed	0.9	
	Countless cracking is observed in external wall	0.9	
	Rain leak without rust of reinforcing bar is observed	0.9	
	No correspondence to the foregoing	1	
Fire experience	Trace	0.7	Structural cracking and deflection Deterioration and aging
	Experience but traceless	0.8	
	No experience	1	
Occupation	Chemical has been used	0.8	Deterioration and aging
	No correspondence to the foregoing	1	
Age of building	30 years or older	0.8	Deterioration and aging
	20 years or older	0.9	
	19 years or less	1	
Finishing condition	Significant spalling of external finishing due to aging is observed	0.9	Deterioration and aging
	Significant spalling and deterioration of internal finishing is observed	0.9	
	No problem	1	

Calculation of Is of Building (Contd...)

Story	Column ID	Column No	C_c	Total C_c	F	α	(n+1)/(n+i)	E_o	T	S_d	I_s
5	C1	4	0.031	0.64	1.00	1.00	0.60	0.39	0.8	1	0.31
	C2	6	0.031								
	C3	4	0.033								
	C4	6	0.033								

Calculation of Is of Building

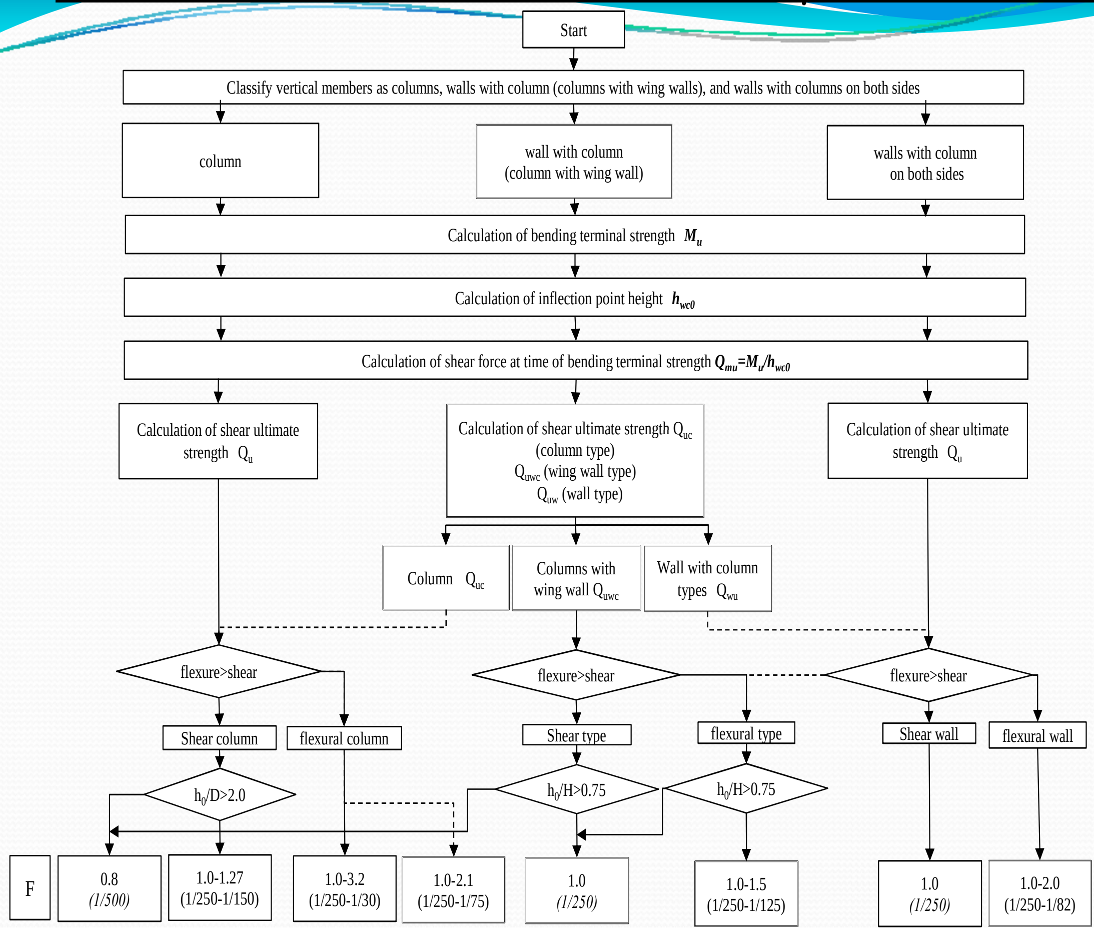
Story	Column ID	Column No	C_c	Total C_c	F	α	(n+1)/(n+i)	E_o	T	S_d	I_s
5	C1	4	0.031	0.64	1.00	1.00	0.60	0.39	0.8	1	0.31
	C2	6	0.031								
	C3	4	0.033								
	C4	6	0.033								
4	C1	4	0.012	0.25	1.00	1.00	0.67	0.17	0.8	1	0.14
	C2	6	0.012								
	C3	4	0.013								
	C4	6	0.013								
3	C1	4	0.008	0.16	1.00	1.00	0.75	0.12	0.8	1	0.10
	C2	6	0.008								
	C3	4	0.008								
	C4	6	0.008								
2	C1	4	0.006	0.12	1.00	1.00	0.86	0.10	0.8	1	0.08
	C2	6	0.006								
	C3	4	0.006								
	C4	6	0.006								
1	C1	4	0.004	0.09	1.00	1.00	1.00	0.09	0.8	1	0.07
	C2	6	0.004								
	C3	4	0.005								
	C4	6	0.005								

2^{nd} LEVEL SCREENING PROCEDURE

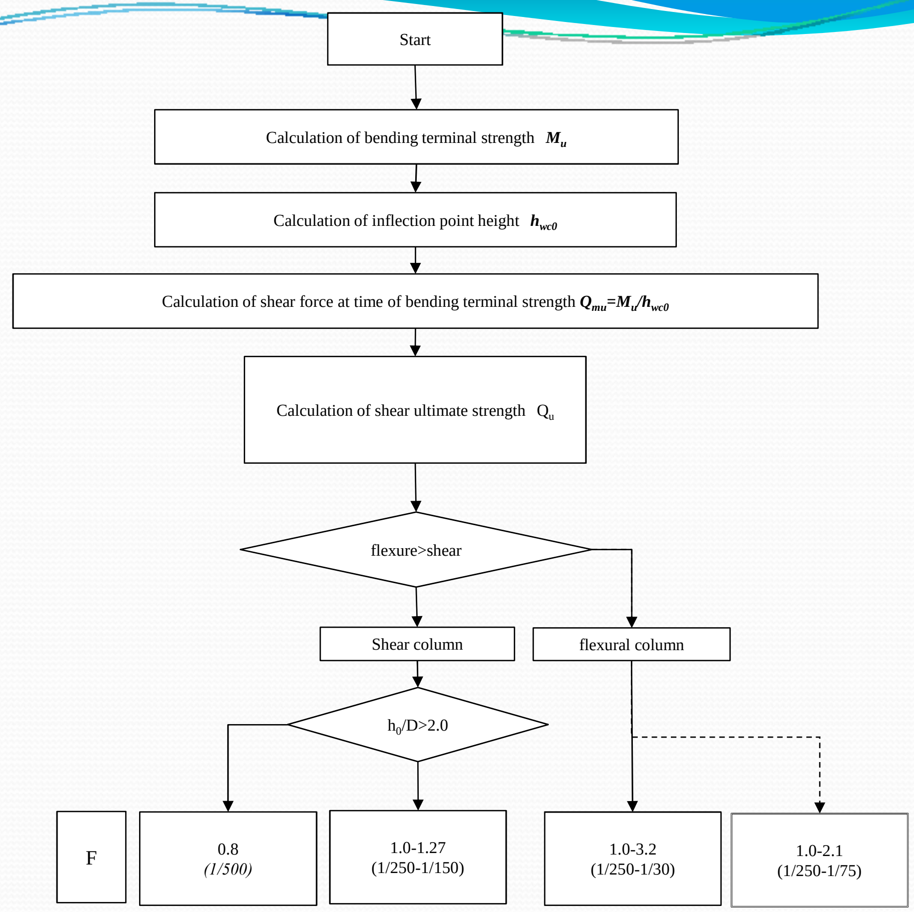
Classification of vertical members based on failure modes in the second level screening

Vertical member	Definition
Shear wall	Walls whose shear failure precede flexural
Flexural wall	Walls whose flexural yielding precede shear failure
Shear column	Columns whose shear failure precede flexural yielding, except for extremely brittle columns
Flexural column	Columns whose flexural yielding precede shear failure
Extremely brittle column	Columns whose h_0/D are equal to or smaller than 2 and shear failure precede flexural yielding

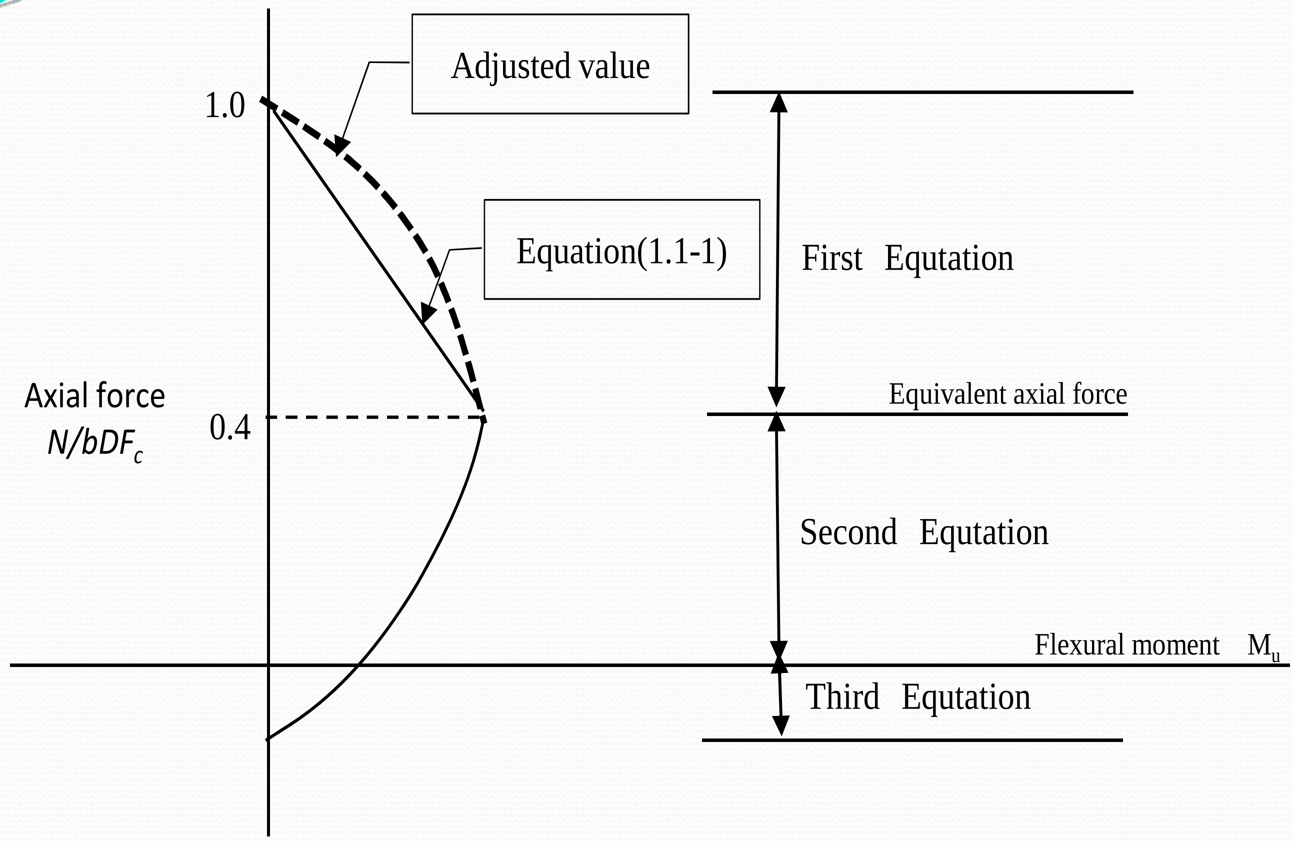
Flow Chart to Calculate The Ductility Index (Contd..)



Flow Chart to Calculate The Ductility Index



Flexural ultimate strength of columns (M_u)



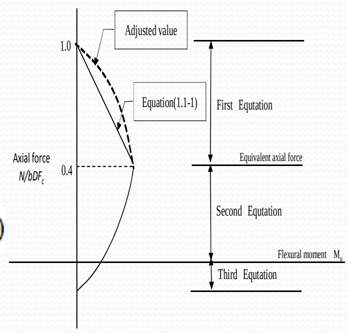
Calculation of Ultimate flexural Strength of Column(M_u)

The ultimate flexural strength of columns shall be calculated

$$M_u = \{0.8a_t\cdot\sigma_y\cdot D + 0.12b\cdot D^2\cdot F_c\}\cdot\left(\frac{N_{max}-N}{N_{max}-0.4b\cdot D\cdot F_c}\right) \quad For\ N_{max} \ge N > 0.4b\cdot D\cdot F_c \quad (1)$$

$$M_u = 0.8a_t\cdot\sigma_y\cdot D + 0.5N\cdot D\cdot\left(1-\frac{N}{b\cdot D\cdot F_c}\right) \quad For\ 0.4b\cdot D\cdot F_c \ge N > 0 \quad (2)$$

$$M_u = 0.8a_t\cdot\sigma_y\cdot D + 0.4N\cdot D \quad For\ 0 > N \ge N_{min} \quad (3)$$



N_{max}	=	Axial compressive strength $= b\cdot D\cdot F_c + a_g\cdot\sigma_y$ (N).
N_{min}	=	Axial tensile strength $= -a_g\cdot\sigma_y$ (N)
N	=	Axial force (N)
a_t	=	Total cross sectional area of tensile reinforcing bars (mm^2)
a_g	=	Total cross sectional area of reinforcing bars (mm^2)
b	=	Column width (mm).
D	=	Column depth (mm).
σ_y	=	Yield strength of reinforcing bars (N/mm^2)
F_c	=	Compressive strength of concrete (N/mm^2)

N=79 KN

N_{max}=2976 KN

0.4bDFc=776 KN

$0.8a_t\sigma_y D$=103.8 KN.m

$0.5ND(1-N/bDF_c)$=0.5*79*375(1-0.041)/1000=14.2 KN.m

$$Q_{mu} = 2\cdot M_u/h_0$$

Mu= 118 KN.m

Q_{mu} = 87 KN

Calculation of Column Load

COLUMN MARK	STORY	AREA(m^2)	N (KN)
C1	5	9.81	79
	4	9.81	200
	3	9.81	320
	2	9.81	441
	1	9.81	562
C2	5	18.37	148
	4	18.37	374
	3	18.37	600
	2	18.37	826
	1	18.37	1052
C3	5	7.88	63
	4	7.88	160
	3	7.88	257
	2	7.88	354
	1	7.88	451
C4	5	14.76	119
	4	14.76	300
	3	14.76	482
	2	14.76	664
	1	14.76	845

Calculation of Ultimate Shear Strength of Column(Q_{su})

$$Q_{su} = \left\{ \frac{0.053 p_t^{\,0.23}(18+F_c)}{M/(Q\cdot d)+0.12} + 0.85\sqrt{p_w \cdot {}_s\sigma_{wy}} + 0.1\sigma_0 \right\} \cdot b \cdot j$$

Q_{su}=149 KN

p_t = Tensile reinforcement ratio (%)

p_w = Shear reinforcement ratio, p_w=0.012 for $p_w \geq 0.012$

${}_s\sigma_{wy}$ = Yield strength of shear reinforcing bars (N/mm^2)

σ_0 = Axial stress in column (N/mm^2)

d = Effective depth of column. D-50mm may be applied.

$\frac{M}{Q}$ = Shear span length. Default value is $\frac{h_0}{2}$

h_0 = Clear height of the column

j = Distance between centroids of tension and compression forces, default value is 0.8D.

If $M/(Q\cdot d)$ is less than unity or greater than 3, the value of $M/(Q\cdot d)$ shall be unity or 3 respectively

if the value of σ_0 is greater than 8N/mm^2, the value of σ_0 shall be 8N/mm^2

Failure Mode Categorization According to the Strength Margin (Contd…)

Story	Column	N(KN)	Mu (KN.m)	Qmu (KN)	Qsu (KN)	Qu (KN)	Type of Column
5	C1	79	118	87	149	87	Flexural
4		200	137	102	159	102	Flexural
3		321	154	114	169	114	Flexural
2		442	226	167	185	167	Flexural
1		562	237	175	194	175	Flexural

Failure Mode Categorization According to the Strength Margin

Story	Column	N(KN)	Mu (KN.m)	Qmu (KN)	Qsu (KN)	Qu (KN)	Type of Column
5	C1	79	118	87	149	87	Flexural
4		200	137	102	159	102	Flexural
3		321	154	114	169	114	Flexural
2		442	226	167	185	167	Flexural
1		562	237	175	194	175	Flexural
5	C2	148	129	96	155	96	Flexural
4		375	160	119	173	119	Flexural
3		601	181	134	191	134	Flexural
2		828	245	181	216	181	Flexural
1		1054	224	166	234	166	Flexural
5	C3	64	92	68	145	68	Flexural
4		161	105	78	152	78	Flexural
3		258	117	87	160	87	Flexural
2		355	174	129	175	129	Flexural
1		452	183	135	183	135	Flexural
5	C4	119	100	74	149	74	Flexural
4		301	122	90	164	90	Flexural
3		483	139	103	178	103	Flexural
2		666	197	146	200	146	Flexural
1		848	202	150	214	150	Flexural

Ductility index of flexural column (F)

In case $R_{mn} < R_y$

$$F = 1.0 + 0.27\frac{R_{mu} - R_{250}}{R_y - R_{250}}$$

In case $R_{mn} \geq R_y$

$$F = \frac{\sqrt{2R_{mu}/R_y - 1}}{0.75 \cdot (1 + 0.05R_{mu}/R_y)} \leq 3.2$$

Where:

R_y = Yield deformation in terms of inter-story drift angle, which in principle shall be taken as R_y=1/150.

R_{250} = Standard inter-story drift angle, R_{250} = 1/250.

R_{mu} = Inter-story drift angle at the ultimate deformation capacity in flexural failure of the column.

Calculation of Upper limit of the drift angle of flexural column (${}_cR_{max}$)

Calculation of drift angle for shear force

$$ {}_cR_{\max(s)} = {}_cR_{250} \quad for \quad {}_c\tau_u / F_c > 0.2 $$

$$ {}_cR_{\max(s)} = {}_cR_{30} \quad for\ other\ case $$

${}_cR_{max(s)}$= 1/30

${}_c\tau_u$ = Shear stress at the column strength.

$$ = \min\left\{ {}_cQ_{mu} / (b \cdot j),\ {}_cQ_{su} / (b \cdot j) \right\}. $$

${}_cQ_{mu}$ = Shear force at the ultimate flexural strength of the column.

${}_cQ_{su}$ = Ultimate shear strength of the column

j = Distance between the centroids of the tension and compression forces. Default value is $0.8D$.

Calculation of drift angle for clear height

$$ {}_cR_{\max(h)} = {}_cR_{250} \quad for \quad h_o / D \le 2 $$

$$ {}_cR_{\max(h)} = {}_cR_{30} \quad for\ other\ case $$

${}_cR_{max(h)}$= 1/30

D = Column depth.

h_o = Clear height of the column.

Calculation of drift angle for Spacing of hoops

$$ {}_cR_{\max(b)} = {}_cR_{50} \quad for \quad s / d_b > 8 $$

$$ {}_cR_{\max(b)} = {}_cR_{30} \quad for\ other\ case $$

${}_cR_{max(b)}$= 1/50

s = Spacing of hoops.

d_b = Diameter of the flexural reinforcing bar of the column.

Calculation of drift angle for tensile reinforcement

$$_cR_{\max(t)} = {_cR_{250}} \quad for \quad p_t > 1.0\%$$

$$_cR_{\max(t)} = {_cR_{30}} \quad for\ other\ case$$

$_cR_{max(t)}$= 1/30

p_t = Tensile reinforcement ratio (%).

Calculation of drift angle for axial force

$$_cR_{\max(n)} = {_cR_{250}} \quad for \quad \eta > \eta_H$$

$$_cR_{max(n)} = {_cR_{30}} \cdot \left(\frac{_cR_{250}}{_cR_{30}} \right)^{n'} \leq {_cR_{30}} \quad for\ other\ case$$

where:

$_cR_{max(n)}$= 1/30

$$n' = (\eta - \eta_L)(\eta_H - \eta_L)\,.$$

$$\eta = N_s / (b \cdot D \cdot F_c)\,.$$

$$\eta_L = 0.25 \quad \text{and} \quad \eta_H = 0.5 \quad \text{for} \quad s \leq 100mm\,.$$

$$\eta_L = 0.2 \quad \text{and} \quad \eta_H = 0.4 \quad \text{for} \quad s > 100mm\,.$$

Calculation of $_cR_{max}$

$$_cR_{\max} = \min\{ {_cR_{\max(n)}}, {_cR_{\max(s)}}, {_cR_{\max(t)}}, {_cR_{\max(b)}}, {_cR_{\max(h)}} \}$$

$_cR_{max(n)}$= 1/30

$_cR_{max(s)}$= 1/30

$_cR_{max(t)}$= 1/30

$_cR_{max(b)}$= 1/50

$_cR_{max(h)}$= 1/30

$_cR_{max}$= 1/50

Yield drift angle of column($_cR_{my}$)

$$_cR_{my} = {}_cR_{150} \quad for \quad h_0 / D \geq 3.0$$

$$_cR_{my} = {}_cR_{250} \quad for \quad h_0 / D \leq 2.0$$

$_cR_{my}$ is set by interporation for $2.0 < h_0 / D < 3.0$

$_cR_{my}$ shall not be greater than that of $_cR_{max}$

$_cR_{my}$= 0.0067=1/150

Inter story drift angle at the flexural yielding of column(R_{my})

$$R_{my} = (h_0 / H_0) \cdot {}_cR_{my} \geq R_{250}$$

where, $h_0 / H_0 \leq 1.0$

R_{my}= 0.0067=1/150

Plastic drift angle of column ($_cR_{mp}$)

$$_cR_{mp} = 10({}_cQ_{su} / {}_cQ_{mu} - q) \cdot {}_cR_{my} \geq 0$$

$$q = 1.0 \quad for \quad s \leq 100mm$$

$$q = 1.1 \quad for \quad s > 100mm$$

$_cR_{mp}$= 0.0407=1/25

$_cQ_{su}$ = Ultimate shear strength of the column.

$_cQ_{mu}$ = Shear force at the ultimate flexural strength of the column.

$_cR_{my}$ = Yield drift angle of column.

S = Spacing of hoops

Inter story drift angle at the ultimate flexural strength of column (R_{mu})

$$R_{mu} = (h_0 / H_0) \cdot {}_cR_{mu} \geq R_{250}$$

where, $h_0 / H_0 \leq 1.0$

$${}_cR_{mu} = {}_cR_{my} + {}_cR_{mp} \leq {}_cR_{30}$$

cRmy= 0.0067=1/150

cRmp= 0.0407=1/25

cRmu=cRmy+cRmp=0.0467=1/21 ≤1/30

So, cRmu = 1/30 ≤cRmax=1/50

So Final cRmax=1/50

Rmu =1/50≥1/250,So Rmu= 1/50

${}_cR_{mu}$ = Drift angle at the ultimate flexural strength of column

${}_cR_{mu}$ shall not be larger than ${}_cR_{max}$

Ductility index of flexural column (F)

1) In Case $R_{mu} < R_y$

$$F = 1.0 + 0.27 \frac{R_{mu} - R_{250}}{R_y - R_{250}}$$

F =2.59

2) In Case $R_{mu} \geq R_y$

$$F = \frac{\sqrt{2R_{mu} / R_y - 1}}{0.75 \cdot (1 + 0.05 R_{mu} / R_y)} \leq 3.2$$

Where:

R_y = Yield deformation in terms of inter-story drift angle, which in principle shall be taken as R_y=1/150.

R_{250} = Standard inter-story drift angle, R_{250} = 1/250.

R_{mu} = Inter-story drift angle at the ultimate deformation capacity in flexural failure of the column.

Calculation of Ductility Index(F) (Contd.)

Story	Column	N (KN)	Mu (KN.m)	Qmu (KN)	Qsu (KN)	Qu (KN)	Type of Column	cRmax	cRmy	cRmp	cRmu	F
5	**C1**	79	118	87	149	87	Flexural	50	150	25	50	2.59
4		200	137	102	159	102	Flexural	50	150	32	50	2.59
3		321	154	114	169	114	Flexural	50	150	39	50	2.59
2		442	226	167	185	167	Flexural	250	250	6604	250	1.00
1		562	237	175	194	175	Flexural	250	250	3141	250	1.00

Calculation of Ductility Index(F)

Story	**Column**	**N (KN)**	**Mu (KN.m)**	**Qmu (KN)**	**Qsu (KN)**	**Qu (KN)**	**Type of Column**	**cRmax**	**cRmy**	**cRmp**	**cRmu**	**F**
5	**C1**	79	118	87	149	87	Flexural	50	150	25	50	2.59
4		200	137	102	159	102	Flexural	50	150	32	50	2.59
3		321	154	114	169	114	Flexural	50	150	39	50	2.59
2		442	226	167	185	167	Flexural	250	250	6604	250	1.00
1		562	237	175	194	175	Flexural	250	250	3141	250	1.00
5	**C2**	148	129	96	155	96	Flexural	50	150	29	50	2.59
4		375	160	119	173	119	Flexural	50	150	42	50	2.59
3		601	181	134	191	134	Flexural	50	150	47	50	2.59
2		828	245	181	216	181	Flexural	250	250	278	250	1.00
1		1054	224	166	234	166	Flexural	250	250	82	250	1.00
5	**C3**	64	92	68	145	68	Flexural	50	150	15	50	2.59
4		161	105	78	152	78	Flexural	50	150	17	50	2.59
3		258	117	87	160	87	Flexural	50	150	20	50	2.59
2		355	174	129	175	129	Flexural	250	250	97	250	1.00
1		452	183	135	183	135	Flexural	250	250	100	250	1.00
5	**C4**	119	100	74	149	74	Flexural	50	150	16	50	2.59
4		301	122	90	164	90	Flexural	50	150	21	50	2.59
3		483	139	103	178	103	Flexural	50	150	24	50	2.59
2		666	197	146	200	146	Flexural	250	250	94	250	1.00
1		848	202	150	214	150	Flexural	250	250	76	250	1.00

Calculation of Strength Index(C)

The strength index C in the second level screening procedure shall be calculated by the following equation

$$C = \frac{Q_u}{\sum W}$$

C=0.16

Where:

Q_u = Ultimate lateral load-carrying capacity of the vertical members in the story concerned.

$\sum W$= The weight of the building including live load for seismic calculation supported by the story concerned.

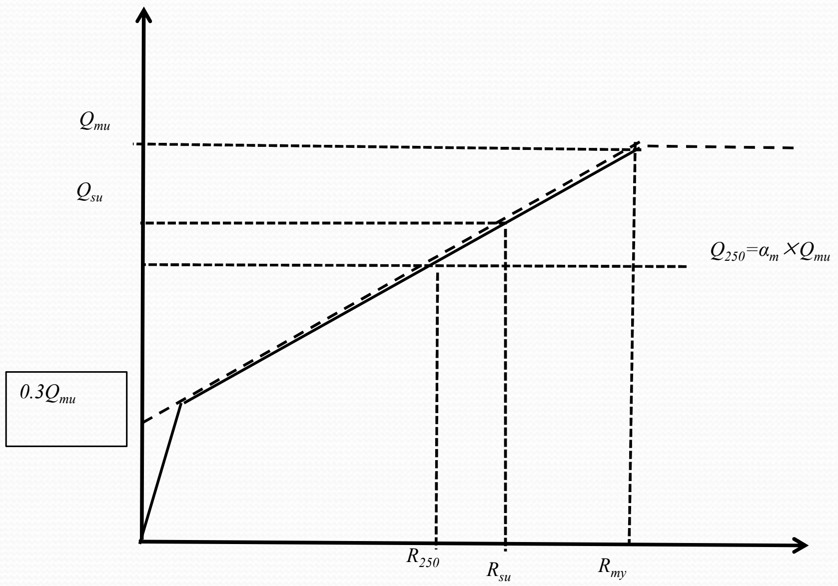
Calculation of Strength Index(C) (Contd…)

Story	Column ID	Qmu (KN)	Qsu (KN)	Qu (KN)	Column No	W (KN)	∑W (KN)	C
5	C1	87	149	87	4	79	2175	0.16
	C2	96	155	96	6	148		0.26
	C3	68	145	68	4	64		0.13
	C4	74	149	74	6	119		0.20

Calculation of Strength Index(C)

Story	Column	Qmu (KN)	Qsu (KN)	Qu (KN)	Column No	W (KN)	∑W (KN)	C
5	C1	87	149	87	4	79	2175	0.16
	C2	96	155	96	6	148		0.26
	C3	68	145	68	4	64		0.13
	C4	74	149	74	6	119		0.20
4	C1	102	159	102	4	200	5498	0.07
	C2	119	173	119	6	375		0.13
	C3	78	152	78	4	161		0.06
	C4	90	164	90	6	301		0.10
3	C1	114	169	114	4	321	8822	0.05
	C2	134	191	134	6	601		0.09
	C3	87	160	87	4	258		0.04
	C4	103	178	103	6	483		0.07
2	C1	167	185	167	4	442	12145	0.06
	C2	181	216	181	6	828		0.09
	C3	129	175	129	4	355		0.04
	C4	146	200	146	6	666		0.07
1	C1	175	194	175	4	562	15468	0.05
	C2	166	234	166	6	1054		0.06
	C3	135	183	135	4	452		0.03
	C4	150	214	150	6	848		0.06

Calculation of Effective Strength Factor(α) Contd…



Relationship between the horizontal force and the displacement

$$\alpha_m = 0.3+0.7\times R_1/R_{my}$$

α_m = Effective strength factor of a flexural column

R_{my} = Drift angle at flexural yielding.

Calculation of Effective Strength Factor(α) Contd...

				Effective Strength Factor(α)			
				R1=R500	R1=R250	R250<R1<R150	R1>R150
Story	**Column ID**	**F index**	**$1/R_{my}$**	**F1(=0.8)**	**F1(=1)**	**F1(1<F1<1.27)**	**F1(1.27<=F1)**
5	C1	2.59	150				1.0
	C2	2.59	150				1.0
	C3	2.59	150				1.0
	C4	2.59	150				1.0

Calculation of Effective Strength Factor(α)

				Effective Strength Factor(α)			
				R1=R500	R1=R250	R250<R1<R150	R1>R150
Story	**Column**	**F index**	**$1/R_{my}$**	**F1(=0.8)**	**F1(=1)**	**F1(1<F1<1.27)**	**F1(1.27<=F1)**
5	C1	2.59	150				1.0
	C2	2.59	150				1.0
	C3	2.59	150				1.0
	C4	2.59	150				1.0
4	C1	2.59	150				1.0
	C2	2.59	150				1.0
	C3	2.59	150				1.0
	C4	2.59	150				1.0
3	C1	2.59	150				1.0
	C2	2.59	150				1.0
	C3	2.59	150				1.0
	C4	2.59	150				1.0
2	C1	1.00	250		1.0		
	C2	1.00	250		1.0		
	C3	1.00	250		1.0		
	C4	1.00	250		1.0		
1	C1	1.00	250		1.0		
	C2	1.00	250		1.0		
	C3	1.00	250		1.0		
	C4	1.00	250		1.0		

Calculation of C_{TU} Indices

				Effective Strength Factor(α)						
				R1=R500	R1=R250	R250<R1<R150	R1>R150			
Story	**Column**	**C**	**F**	**F1(=0.8)**	**F1(=1)**	**F1(1<F1<1.27)**	**F1(1.27<=F1)**	**C_{TU}**	**C_{TU}Sd**	**Evaluation**
5	C1	0.16	2.59				1.0	0.75	0.75	OK
	C2	0.26	2.59				1.0			
	C3	0.13	2.59				1.0			
	C4	0.20	2.59				1.0			
4	C1	0.07	2.59				1.0	0.36	0.36	OK
	C2	0.13	2.59				1.0			
	C3	0.06	2.59				1.0			
	C4	0.10	2.59				1.0			
3	C1	0.05	2.59				1.0	0.25	0.25	NG
	C2	0.09	2.59				1.0			
	C3	0.04	2.59				1.0			
	C4	0.07	2.59				1.0			
2	C1	0.06	1.00		1.0			0.26	0.26	NG
	C2	0.09	1.00		1.0					
	C3	0.04	1.00		1.0					
	C4	0.07	1.00		1.0					
1	C1	0.05	1.00		1.0			0.20	0.20	NG
	C2	0.06	1.00		1.0					
	C3	0.03	1.00		1.0					
	C4	0.06	1.00		1.0					

Calculation of Eo (Contd…)

				Effective Strength Factor(α)					
				R1=R500	R1=R250	R250<R1<R150	R1>R150		
Story	**Column**	**C**	**F index**	**F1(=0.8)**	**F1(=1)**	**F1(1<F1<1.27)**	**F1(1.27<=F1)**	E_o(Eq1)	E_o(Eq2)
5	C1	0.16	2.59				1.0	1.95	1.01
	C2	0.26	2.59				1.0		
	C3	0.13	2.59				1.0		
	C4	0.20	2.59				1.0		

$$E_o(Eq1)=(C1+\sum\alpha1.C1)*F1$$

$$E_o(Eq2)=Sqrt((C1*F1)^2+....+(Ci*Fi)^2))$$

Calculation of Eo

				Effective Strength Factor(α)				Eo	
				R1=R500	R1=R250	R250<R1<R150	R1>R150		
Story	**Column**	**C**	**F**	**F1(=0.8)**	**F1(=1)**	**F1(1<F1<1.27)**	**F1(1.27<=F1)**	E_o(Eq1)	E_o(Eq2)
5	C1	0.16	2.59				1.0	1.95	1.01
	C2	0.26	2.59				1.0		
	C3	0.13	2.59				1.0		
	C4	0.20	2.59				1.0		
4	C1	0.07	2.59				1.0	0.93	0.49
	C2	0.13	2.59				1.0		
	C3	0.06	2.59				1.0		
	C4	0.10	2.59				1.0		
3	C1	0.05	2.59				1.0	0.65	0.34
	C2	0.09	2.59				1.0		
	C3	0.04	2.59				1.0		
	C4	0.07	2.59				1.0		
2	C1	0.06	1.00		1.0			0.26	0.13
	C2	0.09	1.00		1.0				
	C3	0.04	1.00		1.0				
	C4	0.07	1.00		1.0				
1	C1	0.05	1.00		1.0			0.20	0.10
	C2	0.06	1.00		1.0				
	C3	0.03	1.00		1.0				
	C4	0.06	1.00		1.0				

E_o(Eq1)=(C1+∑α1.C1)*F1

E_o(Eq2)=Sqrt((C1*F1)^2+….+(Ci*Fi)^2))

Calculation of Is(Contd…)

Story	Column	(n+1)/(n+i)	E_0(Eq1)	E_0(Eq2)	E_0	S_d	T	I_s
5	C1	0.60	1.95	1.01	1.95	1	0.8	0.94
	C2							
	C3							
	C4							

Calculation of Is

Story	Column	(n+1)/(n+i)	E_0(Eq1)	E_0(Eq2)	E_0	S_d	T	I_s
5	C1	0.60	1.95	1.01	1.95	1	0.8	0.94
	C2							
	C3							
	C4							
4	C1	0.67	0.93	0.49	0.93	1	0.8	0.50
	C2							
	C3							
	C4							
3	C1	0.75	0.65	0.34	0.65	1	0.8	0.39
	C2							
	C3							
	C4							
2	C1	0.86	0.26	0.13	0.26	1	0.8	0.18
	C2							
	C3							
	C4							
1	C1	1.00	0.20	0.10	0.20	1	0.8	0.16
	C2							
	C3							
	C4							

Overview of Seismic Capacity Evaluation According to Japanese Standard

February 12, 2013

By

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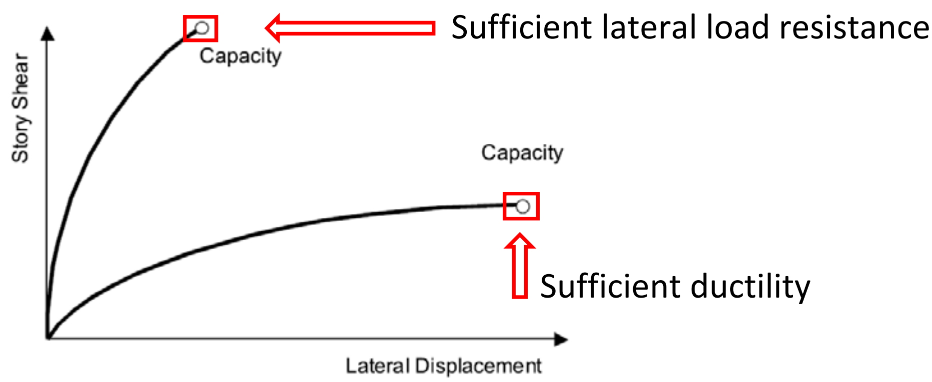
Basic concept of Seismic Resistance:

A structure can resist strong ground motion without collapse if the structure is provided with

- Sufficient **lateral load resistance**
- Limited lateral load resistance with sufficient **ductility.**



Story Shear- Story Drift Relationship

Basic Concept of Seismic evaluation in Japanese Standard

The seismic index of structure *Is*,

$$Is = E_0 * S_D * T$$

Where E_0= Basic seismic index of structure
S_D= Irregularity index
T= Time index

Is should be calculated at each story and in each principal horizontal direction

Basic seismic index of structure E_0

$$E_0 = \phi * C * F$$

Where ϕ= Story Index
C = Strength Index
F = Ductility index

Basic Concept of Seismic Evaluation (Contd.)

The standard consists of **three** different level procedures: first, second and third level procedures.

The first level procedure is the simplest and most conservative procedure. Two major things are considered in this level to calculate the strength:

- Sectional area of columns and walls
- Strength of concrete.

→ Inelastic deformability is neglected in this level.

→ First Level screening should not be used if large eccentricity exists in a floor

Second and Third Level Screening ultimate lateral load carrying capacity of vertical members or frames are evaluated using

material and sectional properties together with

reinforcing details

Characteristics of the ground motion

5

The characteristics of the **ground motion** based on **response spectrum** is expressed by required **Seismic Capacity Index** of Structure, Iso

$$Iso = Es*Z*G*U$$

Where:

Es= Basic Seismic Demand Index of structure

Z = Zone index (Seismic activity at construction area)

G = Ground index (Amplification of ground motion by surface soil deposit)

U = Usage index. (Here in this case 1.5 is used assuming the building will be used as a shelter after a severe earthquake)

First Level Screening Procedure

6

Lateral strength of a story is crudely evaluated by examining the **shear strength** of columns and walls by their cross-sectional areas. The strength of girder is not examined at this stage because:

- The column is believed to be more vulnerable to earthquake force.
- Failures of columns lead to the collapse of the building.
- The girder is believed to be more ductile.

Important Features of First Level Screening

- In order to meet the high seismic design forces, the **Japanese** buildings are provided with **large columns and walls.**
- Importance of **shear wall** is also emphasized in design.

Therefore a Japanese building is believed to possess **lateral strength larger than required by code.**

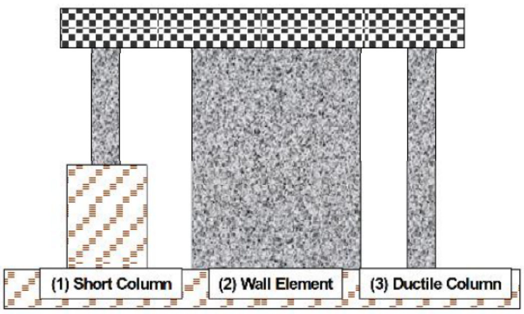
First level screening procedure is to identify these **strong** buildings by a simple calculation.

Vertical Members in First Level Screening

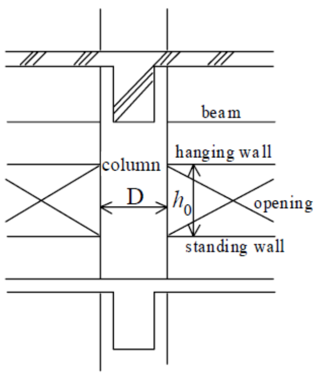
Classification of vertical members in the first level screening

Vertical member	Definition
Column	Columns having *ho/D* larger than 2
Extremely short column	Columns having *ho/D* equal to or less than 2
Wall	Walls including those without boundary columns

Where ***ho*** **: Column clear height**
D **: Column depth**



Idealized building story



Clear height and depth of column

Basic parameters used in First Level Screening

A crude and conservative estimation of **shear strength** per unit sectional area is used for

- short columns 1.5 Mpa
- columns 1.0 Mpa
- walls with boundary columns on both sides 3 Mpa
- walls with boundary columns on one side 2 Mpa
- walls with boundary columns with no boundary 1 Mpa

Based on dimension, materials, reinforcement ratio commonly used in Reinforced Concrete buildings in **Japan.**

Basic Seismic Index for First Level Screening

Short columns are likely to fail in brittle shear mode, and a small ductility index (F=0.8) is assigned.

The **wall and columns** are **assumed** to develop **70% and 50% of their strength**, respectively when the short column fails in shear.

Structural Index E_{0i} of the i story is evaluated by the following equation at the failure of short column:

$$E_{0i}=\underbrace{((n+1)/(n+i))}_{\phi} * \underbrace{(Csc +0.7*Cw +0.5*Cc)}_{C} * \underbrace{0.8}_{F}$$

Basic Seismic Index for First Level Screening (Contd.)

If short column **doesn`t exist** in a story **or** if the **failure of short column** will not lead to the **collapse** of the story, Structural Index E_{0i} of the i story is evaluated by the following equation at the failure of wall:

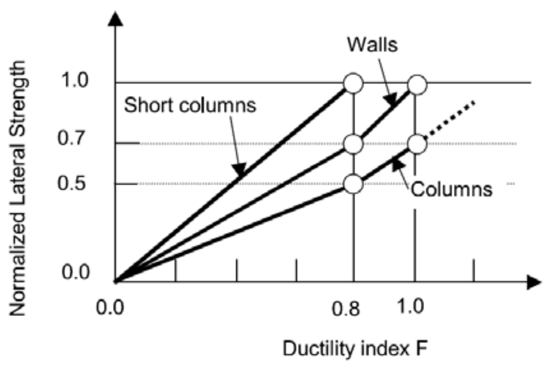
$$E_{0i} = (n+1)/(n+i)\ (C_w + 0.7*C_c)*1.0$$

Where the **ductility index** for wall is selected to be **1.0 and 70% of column strength** is **assumed** to be developed at the failure of wall

Basic Seismic Index for First Level Screening (Contd.)

- If no structural wall/ short column exists in a story (Cw=0), then the structural index is estimated by the following equation:

$$E_{0i} = (n+1)/(n+i)\ C_c*1.0$$



Strength and deformation relation in first level screening procedure

Irregularity/ Configuration Index

Irregularity/ Configuration Index S_D considers the following things:

- Irregularity in plan
- Longitudinal to transverse plan length ratio
- Expansion joints
- Existence of basement
- Abrupt discontinuity of stiffness along the height; especially soft story

A simple grading chart is provided to determine the configuration index which varies from **0.42 to 1.2**

Time/Age Index

In evaluating age index T the following things are to be considered:

- Observed deformation in the building caused by uneven settlement of foundation
- Cracks in columns and walls
- Rust on reinforcement
- Past and present use of chemicals
- Past fire experience
- Finishing condition and building age

Age Index T varies from **0.7 to 1.0**

Second Level Screening Procedure

- The combination of **different ductility levels** and shear resistance of vertical members are considered in earthquake resistance of a structure.
- The **shear resistance** of vertical members (columns and walls) must be calculated on the basis of member geometry, the amount of longitudinal and lateral reinforcement and concrete strength.
- **Failure mode**, either shear or flexural is determined by comparing shear strength and flexural strength

Classification of Vertical Members

- **Classification of vertical members based on failure modes in the second level screening procedure**

Vertical member	Definition	Ductility Index, F
Shear wall	Walls whose shear failure precede flexural yielding	1.0
Flexural wall	Walls whose flexural yielding precede shear failure	1.0-2.0
Shear column	Columns whose shear failure precede flexural yielding, except for extremely brittle columns	1.0
Flexural column	Columns whose flexural yielding precede shear failure	1.27-3.2
Extremely brittle column	Columns whose *ho/D* are equal to or smaller than 2 and shear failure precede flexural yielding	0.8

Dominant Members in Second Level Screening

Ductility-dominant basic seismic index of structure

- Vertical members shall be classified by their ductility indices *F* into **three groups or less**
- The index *F* of the first group shall be taken as **larger than 1.0** and the index *F* of the third group shall be less than the ductility index corresponding to the ultimate deformation of the story
- The **minimum ductility index** of the vertical members should be used in **each group**.

Any grouping of members may be adopted so that the index *E0 would be evaluated as maximum*

Dominant Members in Second Level Screening (Contd.)

$$E_0 = (n+1)/(n+i)\sqrt{(E_1^2 + E_2^2 + E_3^2)}$$

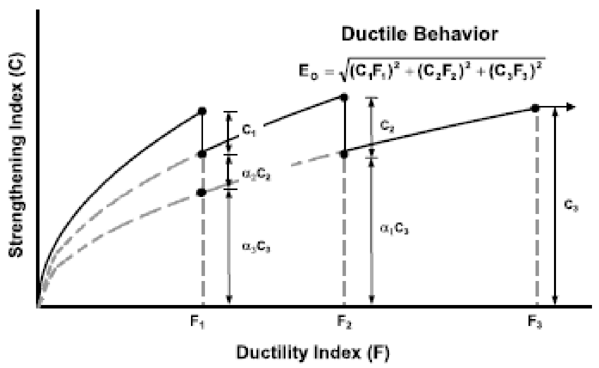
Where

$E_1 = C_1 F_1$

$E_2 = C_2 F_2$

$E_3 = C_1 F_3$

C_1 = The strength index *C* of the first group (with small *F* index).
C_2 = The strength index *C* of the second group (with medium *F* index).
C_3 = The strength index *C* of the third group (with large *F* index).
F_1 = The ductility index *F* of the first group.
F_2 = The ductility index *F* of the second group.
F_3 = The ductility index *F* of the third group.



Ductility dominant members

Dominant Members in Second Level Screening (Contd.)

Strength-dominant basic seismic index of structure

- **ductility index of the first group *F1*** shall be selected as the cumulative point of strength.
- **contribution of strength** indices of only the vertical members with larger ductility indices than that of the first group shall be considered

Any grouping of members may be adopted so that the index E0 would be evaluated as maximum

$$E_0 = (n+1)/(n+i)\,(C_1 + \Sigma\alpha_j C_j) F_1$$

α_j= Effective Strength Factor in the j-th group at the ultimate deformation R_1 corresponding to the first group (Ductility Index F_1)

Dominant Members in Second Level Screening (Contd.)

Effective Strength Factor

Cumulative point of the first group F_1 =0.8 (Drift angle R_1=R_{500}=1/500)		
	F_1	F_1=0.8
	R_1	R_1=R_{500}
Second and higher groups	Shear (R_{su}=R_{250})	α_S
	Shear (R_{250}< R_{su})	α_S
	Flexural (R_{my}=R_{250})	0.65
	Flexural (R_{250}<R_{my}<R_{150})	α_m
	Flexural (R_{my}=R_{150})	0.51
	Flexural and shear walls	0.65

Dominant Members in Second Level Screening (Contd.)

Effective Strength Factor (Contd.)

Cumulative point of the first group $F_1 \geq 1.0$ (Drift angle $R_1 \geq R_{250}=1/250$)				
	F_1	$F_1=1.0$	$1.0<F_1<1.27$	$1.27 \leq F_1$
	R_1	R_{250}	$R_{250}<R_1<R_{150}$	$R_{150} \leq R_1$
Second and higher groups	Shear ($R_{su}=R_{250}$)	1.0	0.0	0.0
	Shear ($R_1<R_{su}$)	α_S	α_S	0.0
	Flexural ($R_{my}<R_1$)	1.0	1.0	1.0
	Flexural ($R_1<R_{my}$)	α_m	α_m	1.0
	Flexural ($R_{my}=R_{150}$)	0.72	α_m	1.0

Dominant Members in Second Level Screening (Contd.)

α_S = Effective strength factor of a shear column, calculated by $\alpha_S = Q_{(F1)}/Q_{su} = \alpha_m \, Q_{mu}/Q_{su} \leq 1.0$

α_m = Effective strength factor of a flexural column, calculated by $\alpha_m = Q_{(F1)}/Q_{mu} = 0.3 + 0.7 \times R_1/R_{my}$

R_{my} = Drift angle at flexural yielding, calculated by Eq. (A1.3-1) in the Supplementary Provisions 1.

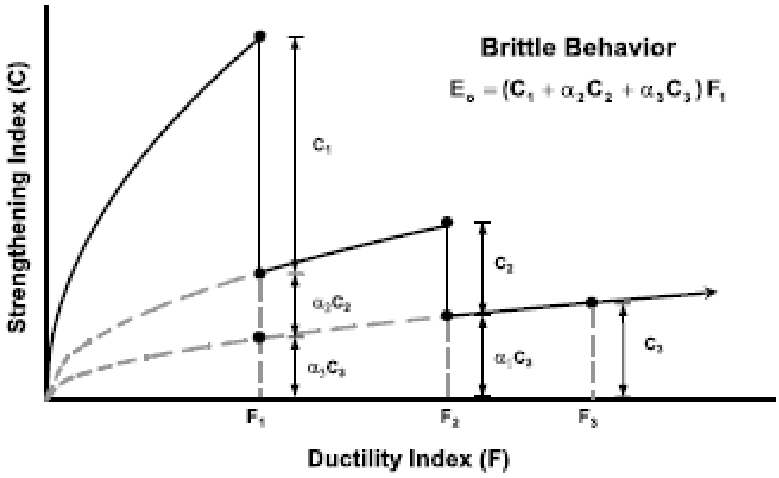
R_{su} = Drift angle at shear strength, calculated by Eq. (A1.2-11) in the Supplementary Provisions 1.

$Q_{(F1)}$ = Shear force at the deformation capacity R_1 of a column in the second and higher groups.

Q_{su} = Shear strength of a column in the second and higher groups (3.2.2).

Q_{mu} = Shear force at flexural yielding of a column in the second and higher groups (3.2.2).

Dominant Members in Second Level Screening (Contd.)



Brittle dominant members

Seismic Index Is after rehabilitation

- If the structural seismic capacity I_S is more than required seismic capacity I_{SO}, the structure is judged safe against earthquake motion observed in **1968 Tokachi Oki earthquake, 1978 Miyagi Ken Oki earthquake or the 1995 Hyogo ken Nanbu** earthquake.
- If Seismic Index I_S is less than index I_{SO} but more than **0.65 I_{SO}** the structure is thought to possess reasonable seismic resistance, but the vulnerability assessment by the second level screening is recommended.

Conclusion

25

Seismic evaluation technique developed in Standard **for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001** is basically based on the existing RCC buildings of Japan, so some parameters may be modified for using it in any other country.

References

26

- Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001, Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001 and Technical Manual for Seismic Evaluation and Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001
- Lecture Notes of IISE, BRI for Earthquake Engineering Course by Shunsuke Sugano, Professor Emeritus, Hiroshima University, Visiting Research Fellow, IISEE, BRI.
- Shunsuke OTANI, Professor, University of Tokyo, "Seismic Vulnerability of Reinforced Concrete Building."
- Toshimi Kabeyasawa, Professor, University of Tokyo, "Improvement of Seismic Performance of Reinforced Concrete School Building in Japan Part1 Damage Survey and Performance Evaluation after 1995 Hyogo- Ken Nambu Earthquake.


Thank You Very Much

SHORT TRAINING COURSE ON SEISMIC ASSESSMENT, RETROFIT DESIGN AND CONSTRUCTION OF RC BUILDING

TITLE OF LECTURE

CONCEPT ON RETROFITTING DESIGN

PRESENTED BY

ANUP KUMAR HALDER

SUB DIVISIONAL ENGINEER

PWDDESIGN DIVISION-V.

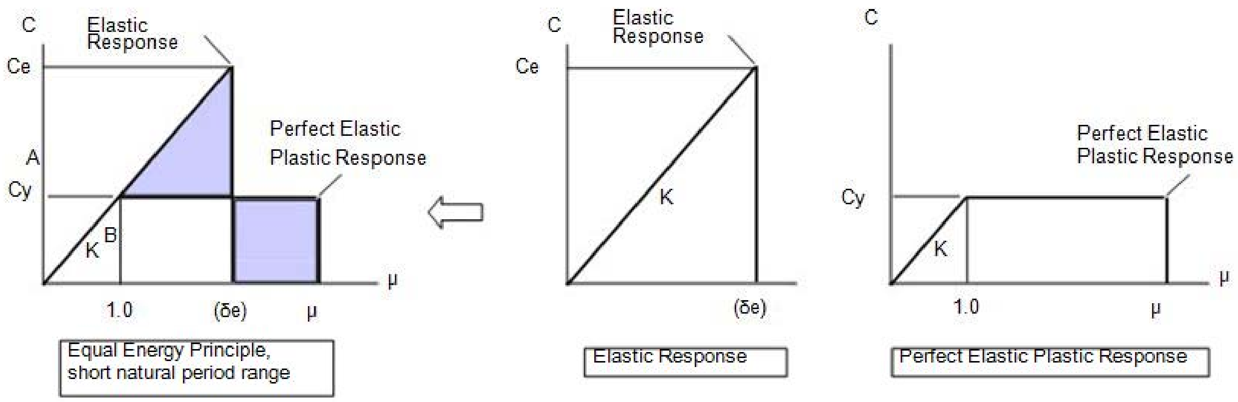
&

TEAM MEMBER WORKING TEAM-II

OUTLINE

1. BASIC CONCEPT
2. SIGNIFICANCE OF “C”
3. SIGNIFICANCE OF “F”
4. SHEAR COLUMN
5. FLEXURAL COLUMN
6. STRENGTH DOMINANT & DUCTILITY DOMINANT STRUCTURE
7. SIGNIFICANCE OF “E_O”
8. ESTABLISHMENT OF“I_{SO}” VALUE
9. IMPORTANCE OF “S_D”
10. IMPORTANCE OF “T”
11. JUDGEMENT OF “I_{SO}” VALUE
12. SEISMIC PERFORMANCE LEVEL AS PER ASCE-41
13. ANALYSIS & ACCEPTANCE CRITERION ASCE-41
14. RETROFITTING METHODS
15. STRENGTHENING EFFECT OBSERVED
16. STRATEGIES & PLANNING

BASIC CONCEPT



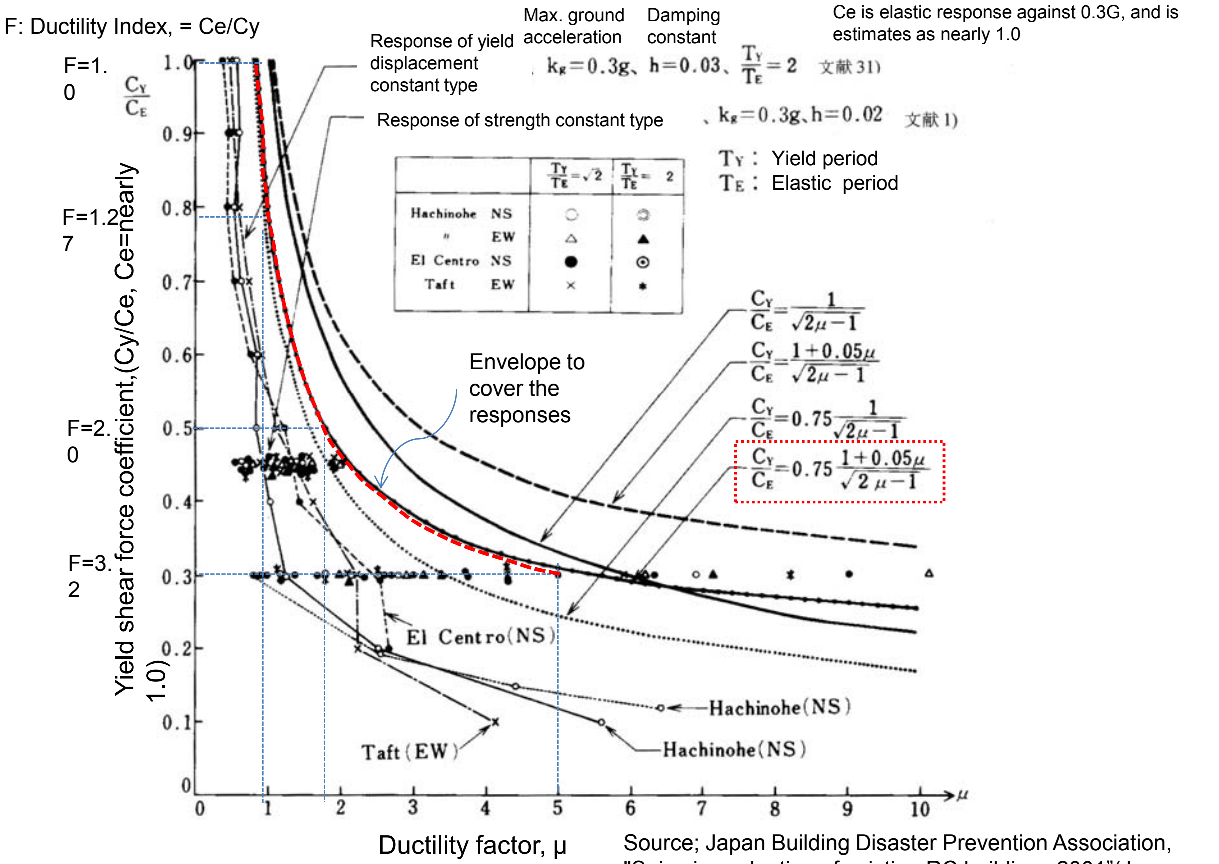
ELASTIC RESPONSE	ELASTIC PLASTIC RESPONSE
GROUND MOTION PRODUCES ELASTIC RESPONSE BASE SHEAR (Ce)	MIN BASE SHEAR COEFFICIENT FOR STUCTURE SYSTEM (Cy)
	DUCTILITY μ (DEFORMATION CAPACITY/YIELD DEFORMATION.)

$C_e = C_y\sqrt{2\mu - 1}$ for short period systems

$C_e = C_y \cdot \mu$ for long period systems

$E_0 = C \cdot F$ FOR A SINGLE DEGREE OF FREEDOM SYSTEM STRUCTURAL RESISTANCE

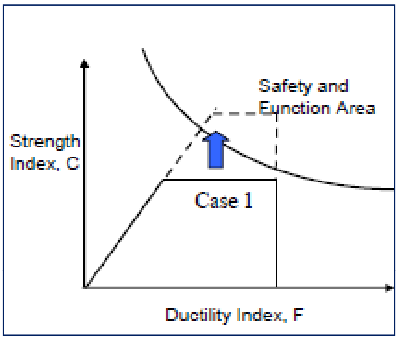
Response ductility factor “μ” and Yield shear force coefficient “Cy/Ce” or ductility index “F (= Ce/Cy)” by Degrading Tri-linear RC frame model



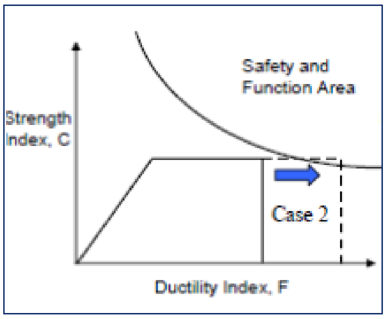
Source; Japan Building Disaster Prevention Association, "Seismic evaluation of existing RC buildings 2001"(Japanese

BASIC CONCEPT cont…

1. INCREASING STRENGTH

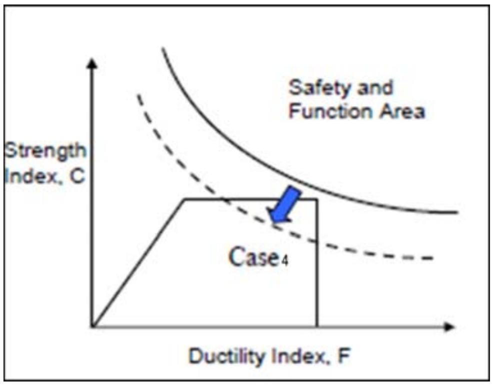


2. INCREASING DUCTILITY



3. INCREASING STRENGTH+ DUCTILITY

4. REDUCTION OF SEISMIC LOAD



5. IMPROVE CONFIGURATION

SIGNIFICANCE OF "C"

- LATERAL STRENGTH OR LOAD CARRYING CAPACITY OF A MEMBER

$$C_C = \frac{\tau_C \cdot A_C}{\Sigma W} \cdot \beta_c \qquad \beta_c = \frac{F_c}{20} \quad F_c \le 20 \qquad \tau_C = 1\ \text{N/mm}^2$$

$$\beta_c = \sqrt{\frac{F_c}{20}} \quad F_c > 20$$

1ST LEVEL

Story	∑W	T. Ac (mm^2)	f'c(Mpa)	(n+1)/(n+i)	β_c	Cc
5	3021	5625000	17	0.60	0.85	1.58
4	7703	5625000	17	0.67	0.85	0.62
3	12386	5625000	17	0.75	0.85	0.39
2	17068	5625000	17	0.86	0.85	0.28
1	21750	5625000	17	1.00	0.85	0.22

$$C = \frac{Q_u}{\sum W}$$

Frame	FL	∑W (KN)	Qu	C
	5	**525**	199	0.3795
	4	**1050**	211	0.2013
	3	**1557**	223	0.1435
	2	**2082**	235	0.1131
2A	1	**2624**	248	0.0943

2ND LEVEL

SIGNIFICANCE OF "F"

- DEFORMATION CAPACITY OF STRUCTURAL MEMBER

Vertical member	Ductility index F
Column ($h_0/D>2$)	1.0
Extremely short column ($h_0/D\leq 2$)	0.8
Wall	1.0

1ST LEVEL

- SHEAR COLUMN

$$F = 1.0 + 0.27\frac{R_{su} - R_{250}}{R_y - R_{250}}$$

- FLEXURAL COLUMN

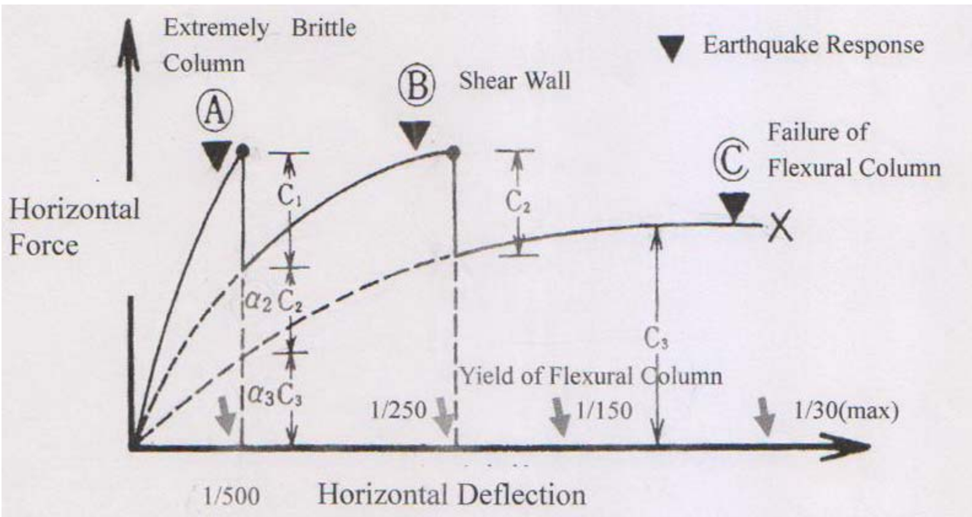
$$F = 1.0 + 0.27\frac{R_{mu} - R_{250}}{R_y - R_{250}} \qquad R_{mu} < R_y$$

$$F = \frac{\sqrt{2R_{mu}/R_y - 1}}{0.75\cdot(1 + 0.05R_{mu}/R_y)} \leq 3.2 \qquad R_{mu} \geq R_y$$

2ND LEVEL

SIGNIFICANCE OF "F" cont.....

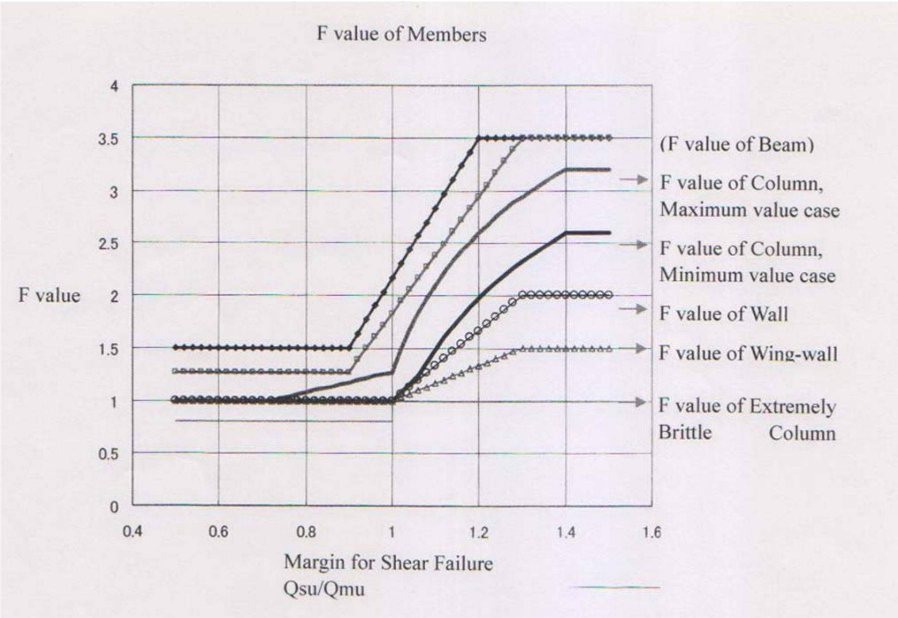
(STANDARD DEFORMATION ANGLE)



Following conditions are assumed in this Code;

Yield deflection storey angle for standard column:	1/150	=0.0066
Ultimate storey deflection angle of shear wall:	1/250	=0.0040
Ultimate storey deflection angle of extremely brittle columns:	1/500	=0.0020

SIGNIFICANCE OF "F" cont.... (Margin of shear failure)



3^{RD} LEVEL

SIGNIFICANCE OF "F" cont.... (Based on Ductility ratio)

DUCTILITY CAPACITY OF A FLEXURAL COLUMN :

$$1 \le \mu = \mu_o - k_1 - k_2 \le 5$$

$$\mu_o = 10\left(\frac{{}_C Q_{su}}{{}_C Q_{mu}} - 1\right)$$

$$k_1 = 2.0$$ (K1=1; WHEN HOOP SPACING 8TIMES THE DIA OF MAIN RE BAR)

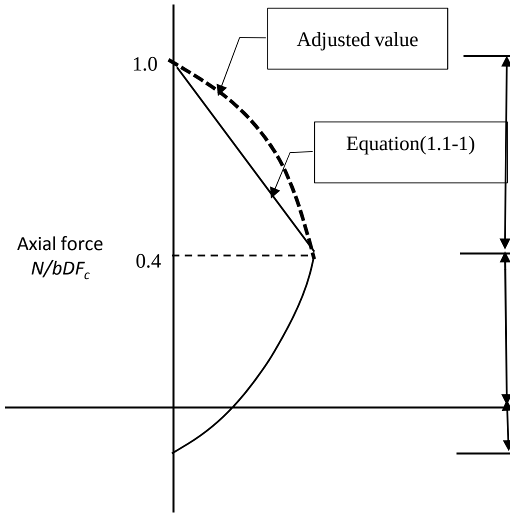
$$k_2 = 30\left(\frac{{}_C \tau_{mu}}{F_c} - 1\right) \ge 0$$

$$_C\tau_{mu} = {}_C Q_{mu} / (b \cdot j)$$

DUCTILITY INDEX F=1; IF FOLLOWING CONDITION IS SATISFIED

$$N_s/(bDF_c) > 0.4$$
$$_C\tau_{mn}/F_c > 0.2$$
$$P_t > 1\%$$
$$h_o/D \le 2.0$$

FLEXURAL COLUMN



First Equation

$$For\ N_{max} \geq N > 0.4b \cdot D \cdot F_c$$

$$M_u = \left\{0.8a_t \cdot \sigma_y \cdot D + 0.12b \cdot D^2 \cdot F_c\right\} \cdot \left(\frac{N_{max} - N}{N_{max} - 0.4b \cdot D \cdot F_c}\right)$$

Equivalent axial force

Second Equation

$$For\ 0.4b \cdot D \cdot F_c \geq N > 0$$

Flexural moment M_u

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.5N \cdot D \cdot \left(1 - \frac{N}{b \cdot D \cdot F_c}\right)$$

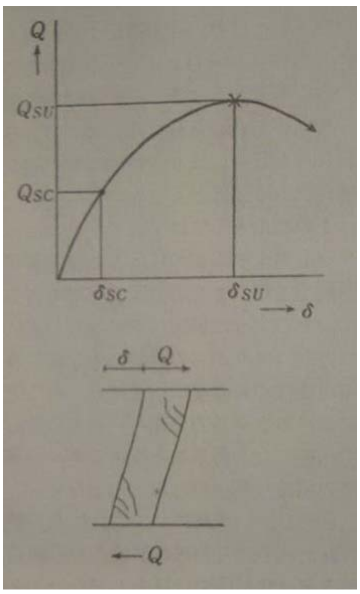
Third Equation

$$For\ 0 > N \geq N_{min}$$

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.4N \cdot D$$

$$Q_{mu} = 2 \cdot M_u / h_0$$

SHEAR COLUMN



FROM EMPIRICAL EQUATION

MAIN REBAR RATIO , CONCRETE STRENGTH

AXIAL FORCE RATIO

$$Q_{su} = \left\{\frac{0.053p_t^{0.23}(18 + F_c)}{M/(Q \cdot d) + 0.12} + 0.85\sqrt{p_w \cdot {}_s\sigma_{wy}} + 0.1\sigma_0\right\} \cdot b \cdot j$$

SLENDERNESS

SHEAR REINFORCEMENT

STRENGTH TYPE & DUCTILITY TYPE STRUCTURE

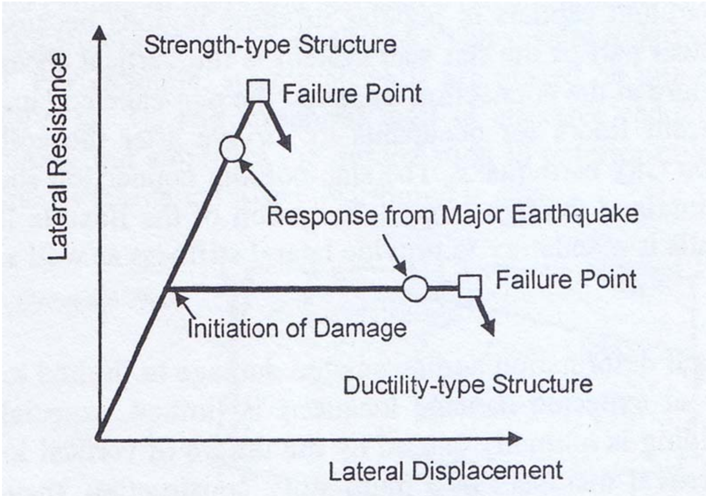
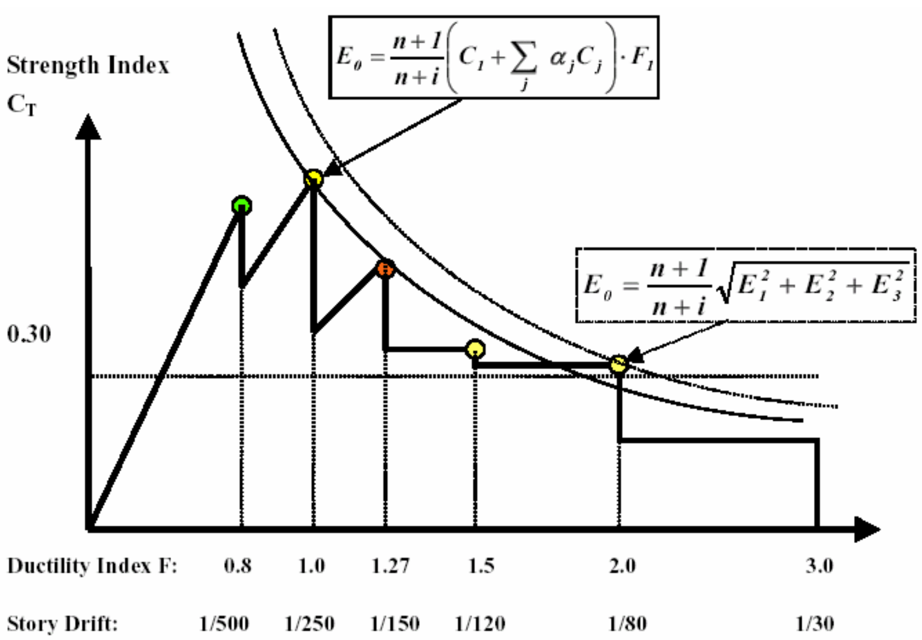


Fig. : Strength and Ductility of a Structure

SOURCE: PROFESSOR SHUNSUKE OTANI'S PAPER

SIGNIFICANCE OF E_0



Idealized relations of lateral strength and ductility for seismic index

SOURCE: PROFESSOR KABAYASAWA'S PAPER

ESTABLISHMENT OF Iso (based on 1st level)

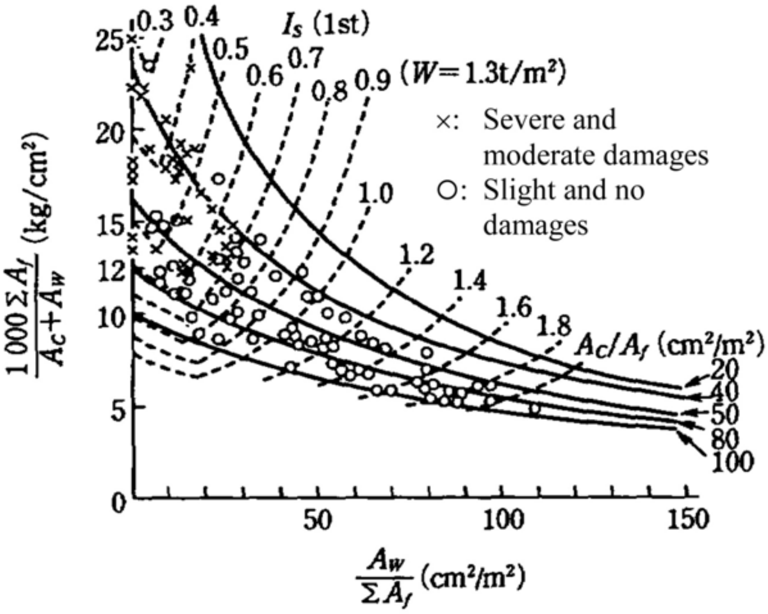
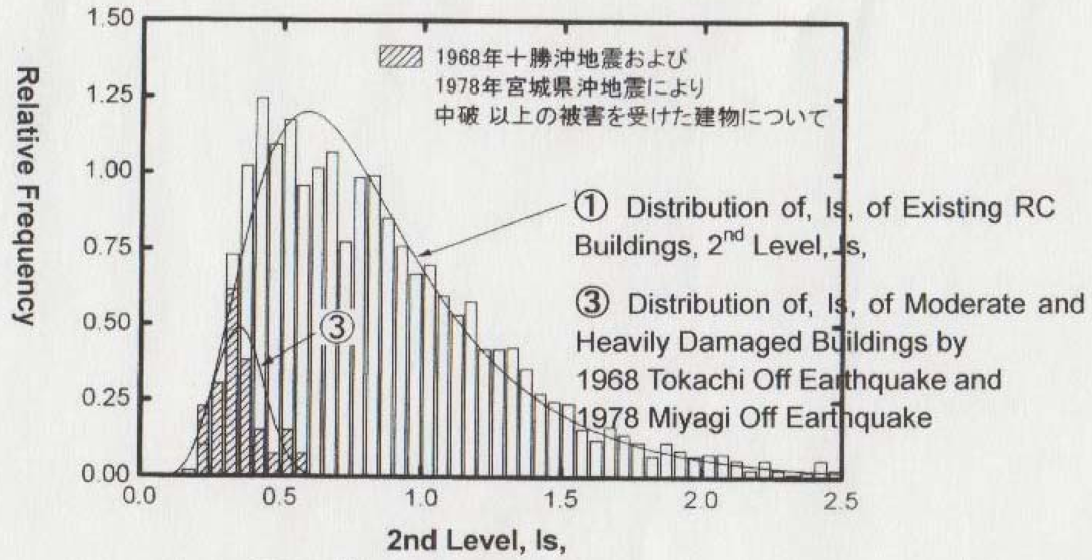


Figure TN.1-3 Index I_S and building damage
(1968 Tokachi-oki and 1978 Miyagiken-oki earthquakes)
(quoted from Figure 4 on page 511 of Ref. 1)

ESTABLISHMENT OF Iso (based on 2nd level)

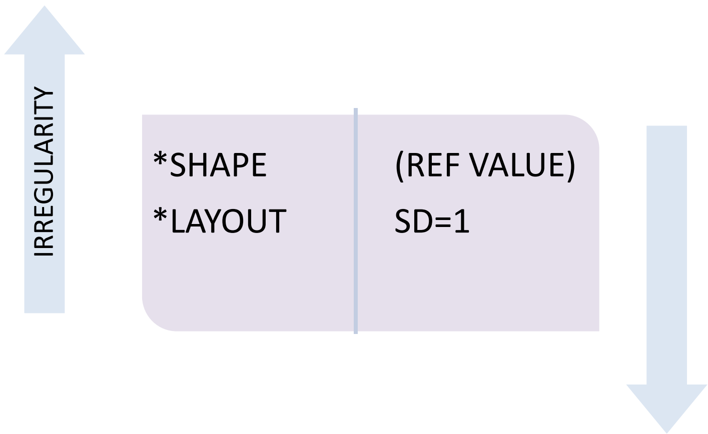


(Figure A5.2-3 of the Japanese Code)

IMPORTANCE OF $\mathbf{S_D}$

IT MODIFY SEISMIC INDEX BY QUANTIFYING THE EFFECT OF

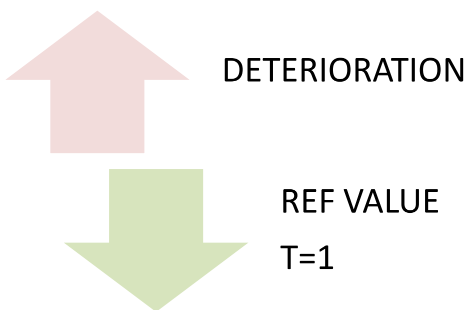
- HORIZONTAL BALANCE
- ELEVATION BALANCE
- ECCENTRICITY
- STIFFNESS



IMPORTANCE OF **T**

TIME INDEX EVALUATES THE EFFECTS OF STRUCURAL DEFECTS

- STRUCTURAL CRACKING AND DEFLECTION
- DETERIORATION AND AGING.



JUDGEMENT

$$I_S \geq I_{SO}$$

where:

I_S = Seismic index of structure

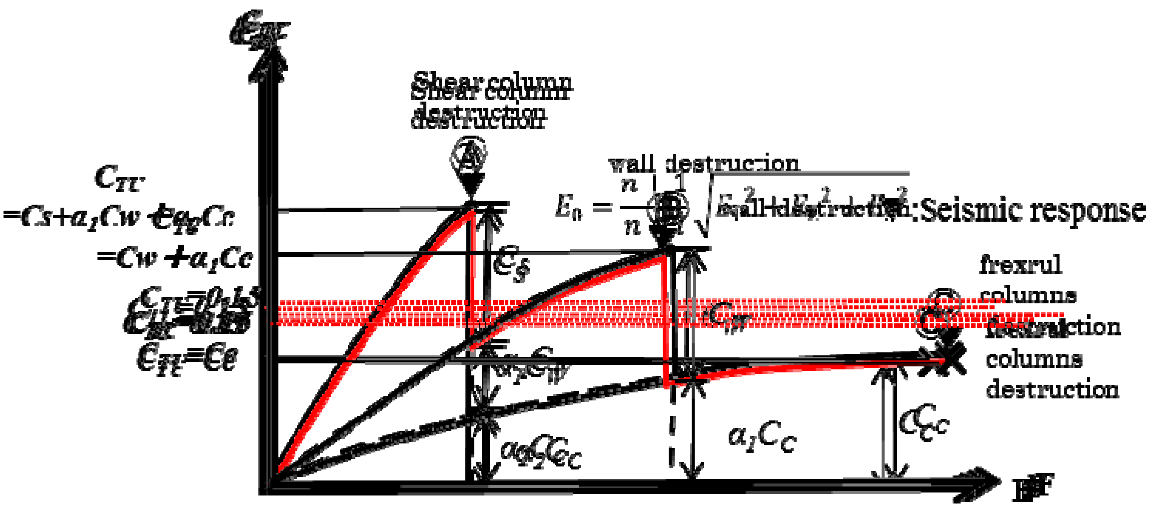
I_{SO} = Seismic demand index of structure

$$C_{TU} \cdot S_D \geq 0.3 \cdot Z \cdot G \cdot U$$

C_{TU} = Cumulative strength index at the ultimate deformation of structure.

S_D = Irregularity index.

JUDGEMENT



JUDGEMENT

$$I_{SO} = E_S \cdot Z \cdot G \cdot U$$

E_S = Basic seismic demand index of structure, standard values of which shall be selected as follows regardless of the direction of the building:

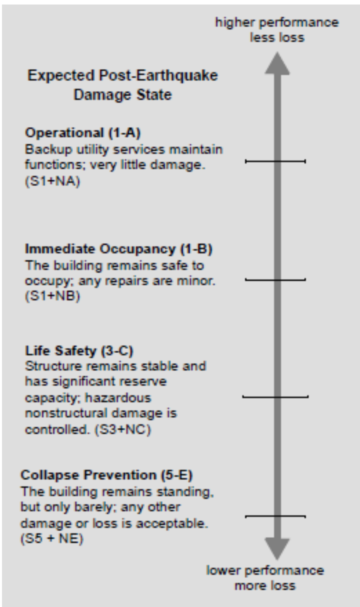
$E_S = 0.8$ for the first level screening,
$E_S = 0.6$ for the second level screening, and
$E_S = 0.6$ for the third level screening.

Z = Zone index, namely the modification factor accounting for the seismic activities and the seismic intensities expected in the region of the site.

G = Ground index, namely the modification factor accounting for the effects of the amplification of the surface soil, geological conditions and soil-and-structure interaction on the expected earthquake motions.

U = Usage index, namely the modification factor accounting for the use of the building.

SEISMIC PERFORMANCE LEVELS



TARGET BUILDINGS PERFORMANCE LEVEL (ASCE-41)

PERFORMANCE LEVEL ASCE 41

Table C1-2. Damage Control and Building Performance Levels

Table C1-3. Structural Performance Levels and Damage[1,2,3]—Vertical Elements

Elements	Type	Structural Performance Levels: Collapse Prevention (S-5)	Life Safety (S-3)	Immediate Occupancy (S-1)
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (< 1/8-in. width) for ductile columns. Minor spalling in nonductile columns. Joint cracks < 1/8 in. wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints < 1/16-in. width.
	Drift	4% transient or permanent.	2% transient; 1% permanent.	1% transient; negligible permanent.

Table C1-4. Structural Performance Levels and Damage[1,2]—Horizontal Elements

Table C1-6. Nonstructural Performance Levels and Damage[1]—Mechanical, Electrical, and Plumbing Systems/Components

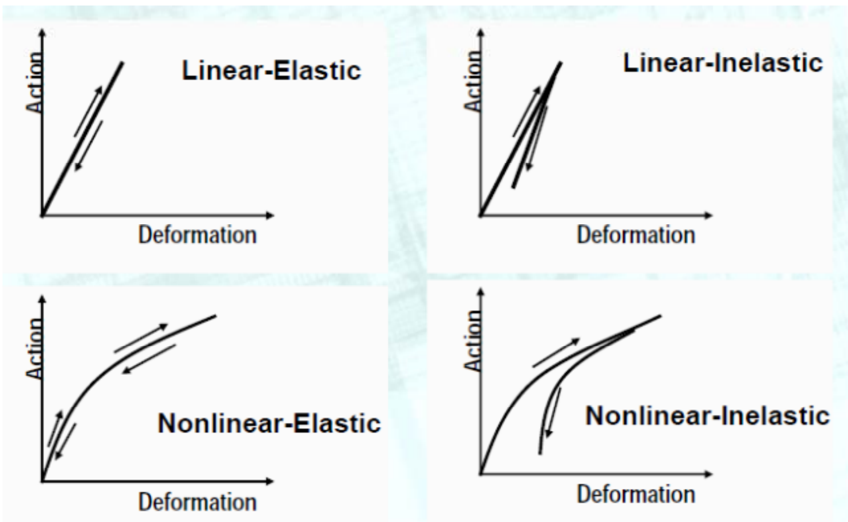
Table C1-5. Nonstructural Performance Levels and Damage[1]—Architectural Components

Table C1-7. Nonstructural Performance Levels and Damage[1]—Contents

Table C1-8. Target Building Performance Levels and Ranges

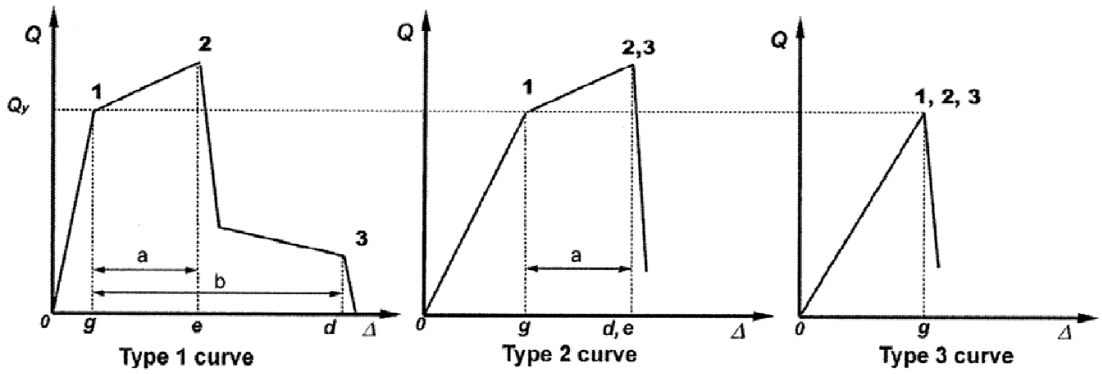
ANALYSIS PROCEDURE (ASCE-41)

- 1. LINEAR STATIC
- 2. LINEAR-DYNAMIC
- 3. NONLINEAR STATIC
- 4. NONLINEAR-DYNAMIC



ACCEPTANCE CRITERIA (ASCE-41)

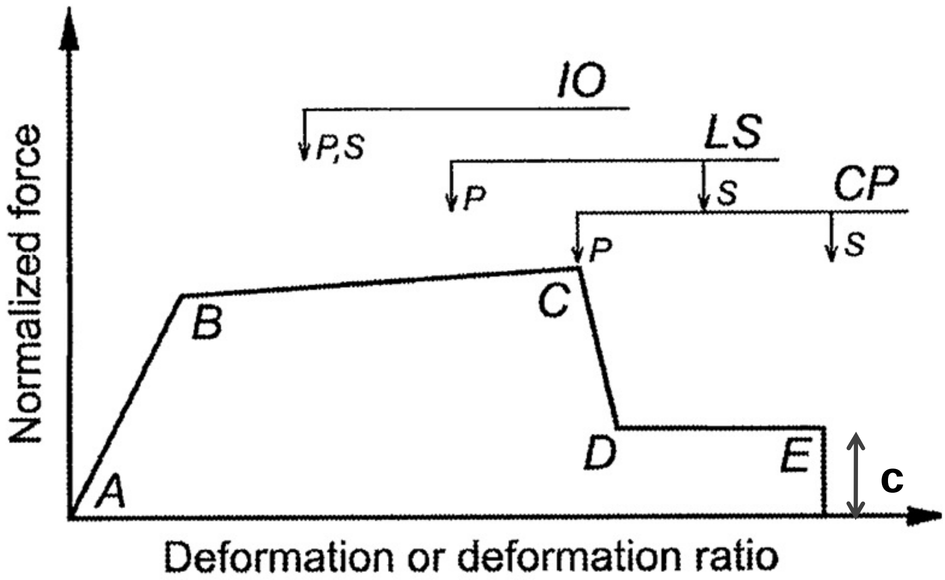
- PRIMARY COMPONENT (P)
- SECONDARY COMPONENT (S)
- DEFORMATION CONTROLLED ACTION
- FORCE CONTROLLED ACTION



DEFORMATION CONTROLLED (e≥2g)

Component Force Versus Deformation Curves.

ACCEPTANCE CRITERIA cont....(ASCE-41)



COMPONENT OR ELEMENT DEFORMATION ACCEPTANCE CRITERIA

NUMERICAL ACCEPTANCE CRITERIA FOR COLUMNS, ASCE-41

Table 6-8 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— Reinforced Concrete Columns

Conditions			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear[1, 3]										
All cases[5]			—	—	—	—	—	—	.0030	.0040
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ d/2			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing > d/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iv. Columns with axial loads exceeding 0.70P$_o$[1, 3]										
Conforming hoops over the entire length			0.015	0.025	0.02	0.0	0.005	0.01	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

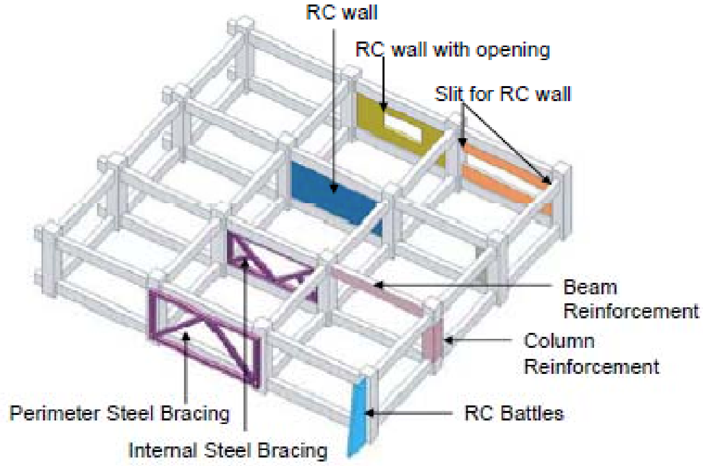
0.0066 (1/150)

0.0040 (1/250)

RETROFITTING METHODS

A. STRENGTH UPGRADING	1. ADDING WALL 2. STEEL WITH FRAME 3. EXTERIOR STEEL FRAME 4. STRUCTURAL FRAME 5. OTHERS
B. DUCTILITY UPGRADING	1. RC JACKETING 2. STEEL JACKETING 3. FRP WRAPING
C. PREVENTION OF DAMAGE CONNECTION	1. IMPROVEMENT OF VIBRATION PROPERTY 2. IMPROVEMENT OF EXTREME BRITTLE MEMBER
D. REDUCTION OF SEISMIC FORCES	1. MASS REDUCTION 2. SEISMIC ISOLATION 3. STRUCTURAL RESPONSE DEVICE
E. STRENGTHENING OF FOUNDATION	1. STRENGTHENING FOUNDATION BEAM 2. STRENGTHENING OF PILE

RETROFITTING METHODS cont..



Building Contractors Society (BCS), Japan
'Seismic Retrofitting Brochure 2006'

STRENGTHENING EFFECT OBSERVED IN STRUCTURAL TEST

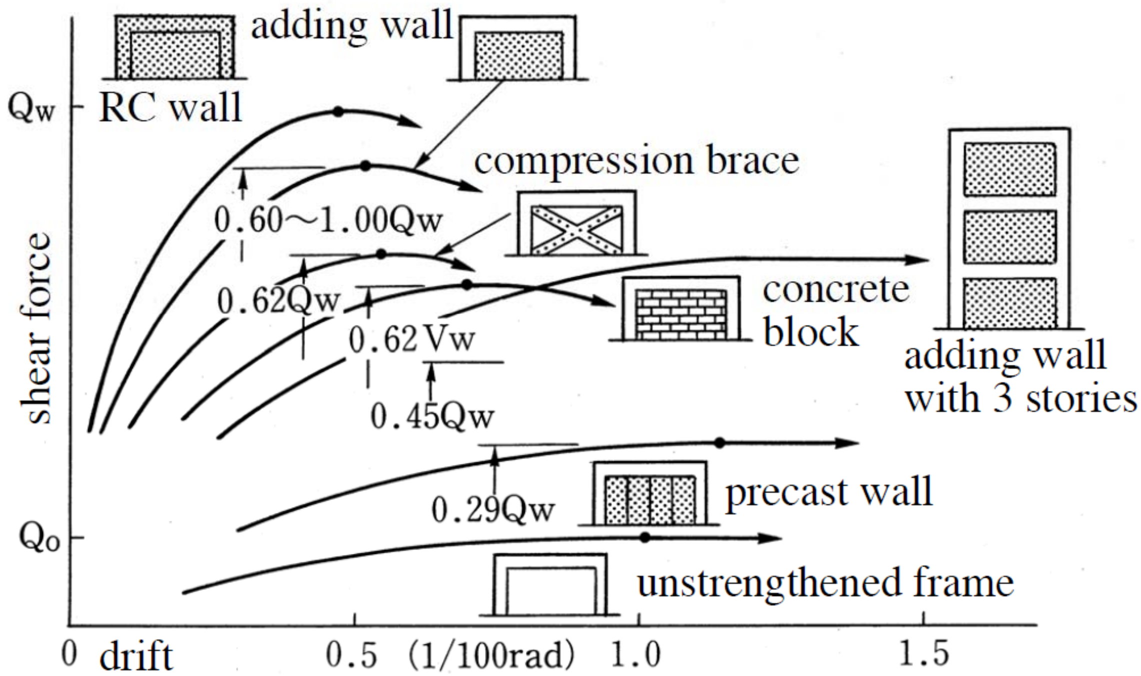
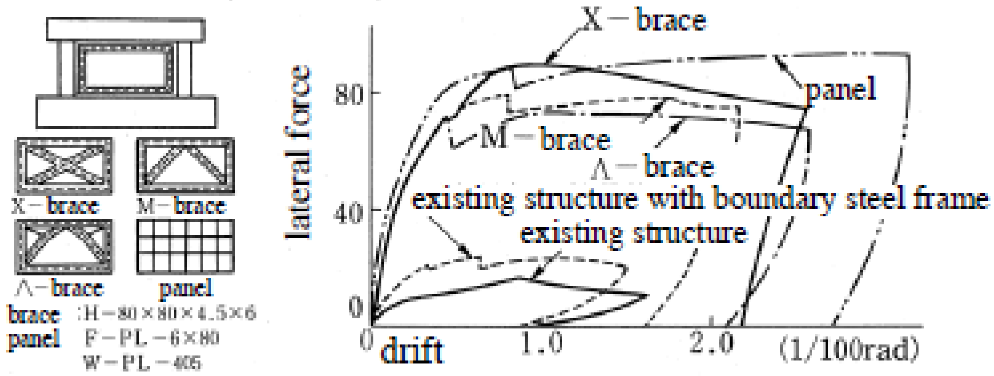


Figure TN.9 Strengthening effect observed in previous structural tests
(quoted from the figure on page 73 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)
(a) strengthening of frame

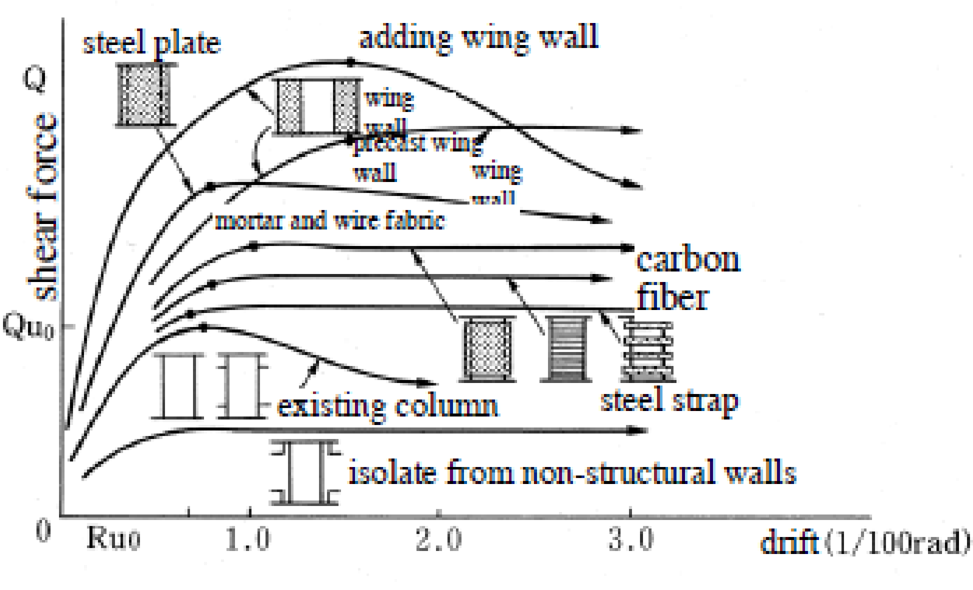
STRENGTHENING EFFECT OBSERVED IN STRUCTURAL TEST cont..



(b) strengthened structure with steel brace with boundary steel frame

Figure TN.9 Strengthening effect observed in previous structural tests
(quoted from the figure on page 73 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)

STRENGTHENING EFFECT OBSERVED IN STRUCTURAL TEST cont….



(c) strengthening of column

Figure TN.9 Strengthening effect observed in previous structural tests
(quoted from the figure on page 73 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)

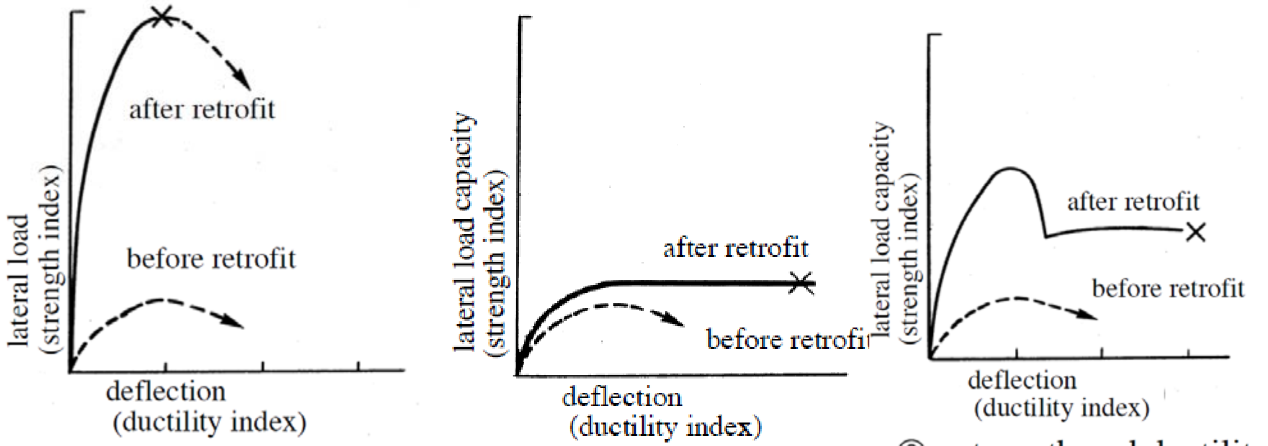
STRATEGIES

- IMPROVING REGULARITIES
- STRENGTHENING
- DUCTILITY
- DAMPING
- MASS REDUCTION
- CHANGING USE

DESIGN PROCEDURE

- PLANNING
- STRUCTURAL DESIGN
- DETAILED DESIGN
- EVALUATION OF RETROFIT EFFECT

PLANNING & STRUCTURAL DESIGN cont..



① strength upgrading ② ductility upgrading ③ strength and ductility

RETROFITTING DESIGN METHODS

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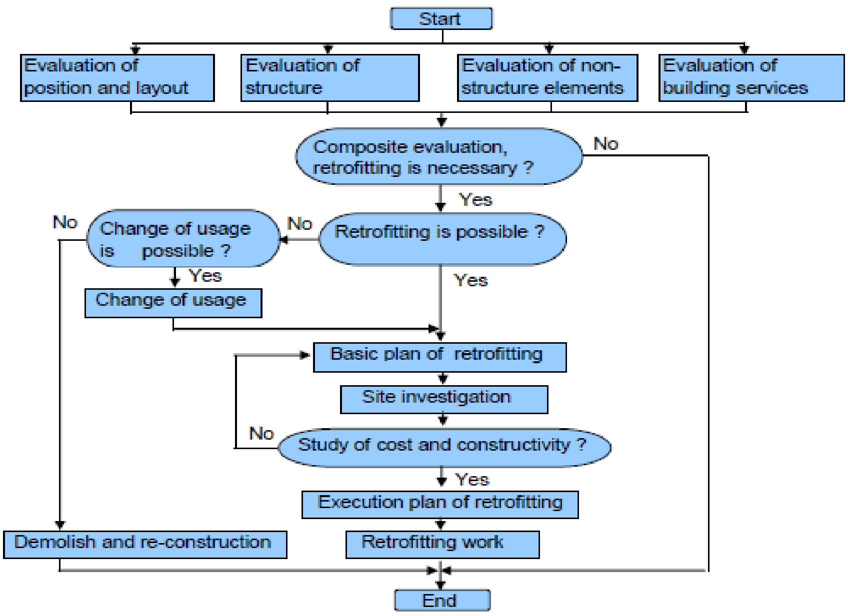
PUBLIC WORKS DEPARTMENT AND
TEAM MEMBER, COMPONENT-2,
CNCRP PROJECT

A flow chart of Seismic Evaluation and Retrofitting for public buildings (facilities), Japan



Source: Building Integrity Center, 1996 "Guideline and explanation of composite seismic evaluation and retrofitting for public facilities (in Japanese)"

Methods of Retrofitting

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Types of Retrofitting Methods				
No.	Description of Retrofitting Methods	Improvement of Strength	Improvement of Ductility	Improvement of Structural Balance
1	Steel Framed Bracing	○		
2	Infilling New RC Shear Wall into Open Frame	○		○
3	Increasing Thickness of Existing Shear Wall	○		○
4	Infilling Steel Plate Wall into Open Frame	○		○
5	Constructing New RC Wing Wall to RC Column	○		
6	Constructing External Frame	○		
7	Constructing External Buttress	○		
8	Steel Plate Jacketing around RC Column		○	
9	Carbon Fiber (Sheet / Strand) Wrapping around RC Column		○	
10	Concrete Jacketing around RC Column	○	○	
11	Providing New Seismic Slit		○	○

Seismic Index of Structure (Is)

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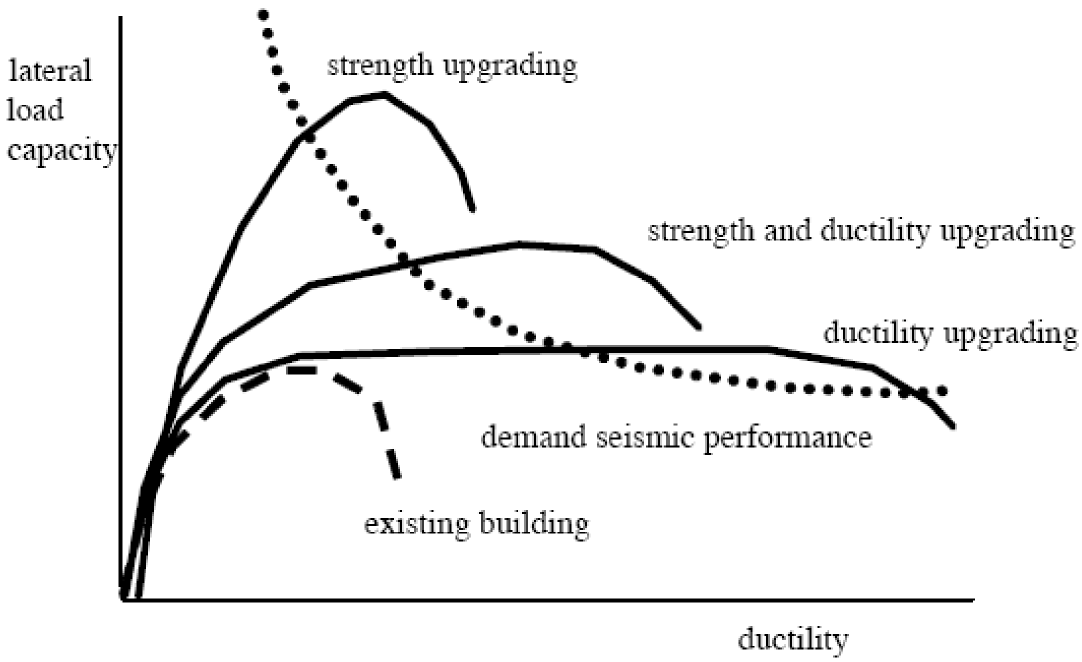

Source: Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001 (English version, 1st edition),

Methods of Retrofitting

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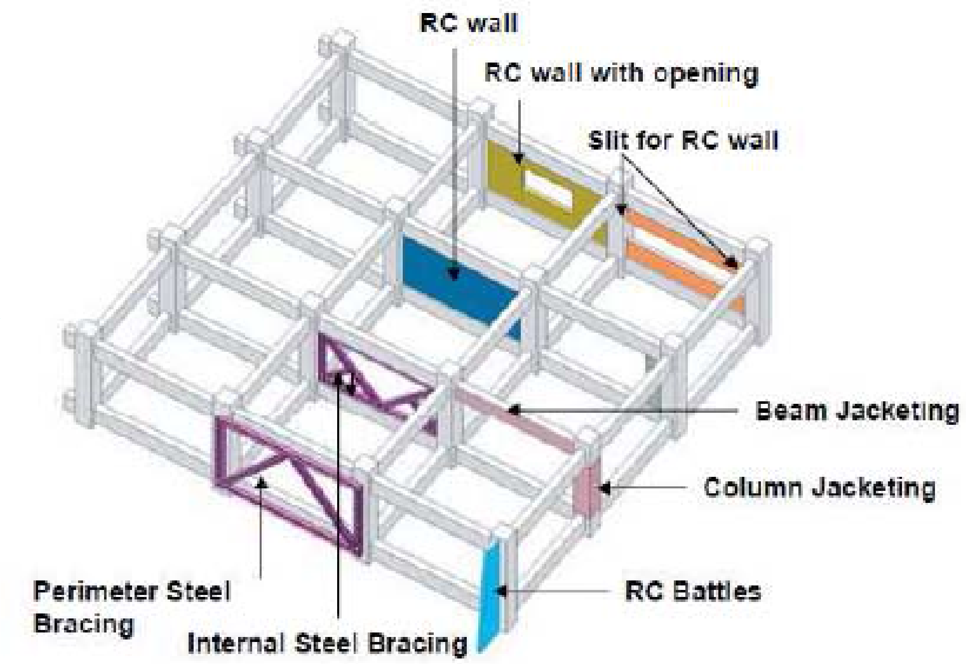

Source: Building Contractors Society (BCS), Japan 'Seismic Retrofitting Brochure 2006'

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Basics of Retrofitting Design

Seismic Index of Structure,

$Is = EoS_DT$ (1)

whrere: **Eo** : Basic Seismic Index of Structure

S_D : Irregularity Index

T : Time Index

Eo as the larger one from eqs (4) and (5). Each equation is calculated within the limitation of the maximum ductility index.

Eo of ductility-dominant Structure,

$Eo = (n+1/n+i) * \sqrt{(C1*F1)^2+(C2*F2)^2+(C3*F3)^2}$ (4)

Eo of strength-dominant Structure,

$Eo = (n+1/n+i) * (C1+\Sigma \alpha j\ Cj)F1$ (5)

whrere: **C** : Strength Index,

F : Ductility Index, Ductility Index is estimated mainly depending on the margin of members against shear failure.

n+1/n+i : Storey-shear modification factor

α : Effective strength factor

$C = Qu / \Sigma W$ (12)

Qu :Ultimate lateral load-carrying capacity of the vertical members in the storey concerned

ΣW :Total weight supported by the storey concerned

Source: Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001 (English version, 1st edition),

Check no -1:

$I_S \geq I_{so}$

I_{So} = Seismic demand index of structure

$= E_S . Z. G. U$

E_s = Basic seismic demand index of structure

Z = Zone index

G = Ground index

U = Usage index

Check no -2:

$C_{TU}. S_D \geq 0.3 . Z . G . U$

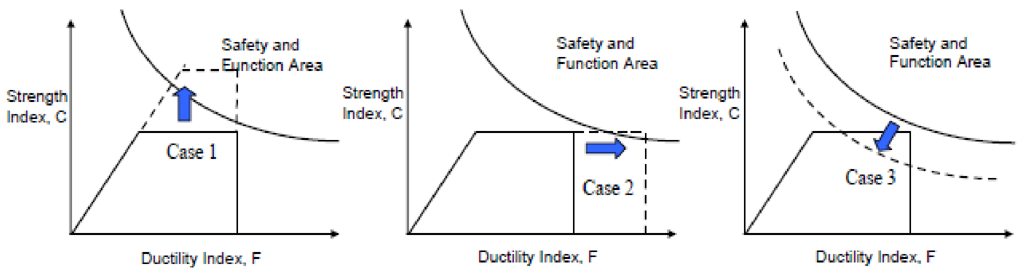
C_{TU} = Cumulative strength index at ultimate deformation of structure

S_D = Irregularity index

Source: Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001 (English version, 1st edition),

Concept of Seismic Retrofitting - Combination of strength and ductility

1 Increasing strength (Case 1 of following figure)
2 Increasing ductility (Case 2 of following figure)
3 Improvement of configuration (Case 3 of following figure)
4 Reduction of seismic load (Case 3 of following figure)



[Source: "Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001", The Japan Building Disaster Prevention Association (English version)]

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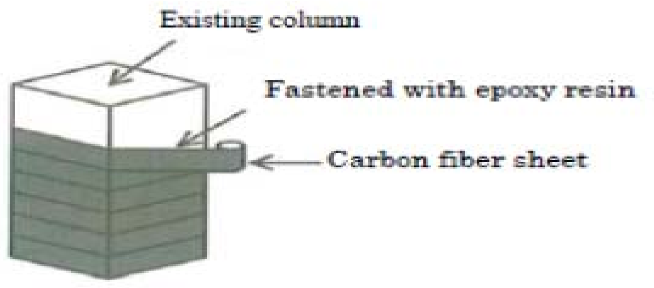
Outline of retrofitting method

1.Carbon Fiber Sheet Wrapping around RC Column

Existing columns in buildings are wrapped with carbon fibre sheets



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STRENGTH OF COLUMN AFTER CARBON FIBER WRAPPING

Shear strength of Column

$Qsu = [0.053\, p_{t2}^{0.23}(F_{c1}+18)/(M/Q\, d+0.12)$

$+0.85\, Sqrt(p_w \sigma_{wy} + p_{wf} \sigma_{fd}) + 0.1\sigma_0]\, b\, j$

p_{t2}= tensile reinforcement ratio of existing column in %

p_w= shear reinforcement ratio of existing column in decimal

p_{wf}= shear reinforcement ratio of carbon fiber sheet in decimal

F_{c1} =compressive strength of concrete for existing structure, N/mm^2

M/ Qd ranges from 1 to 3 and j= 0.8D

b= width of column and D= depth of column

σ_0=axial compressive stress and maximum value 7.8 N/mm^2

d= effective depth of column

σ_{fd}=tensile strength of carbon fiber sheet for shear design

Retrofitting with Carbon Fiber Wrapping

Main features of Carbon Fiber Wrapping:

•Carbon fiber sheet is wrapped with epoxy resin around existing column.

•This method is done for upgrading ductility.

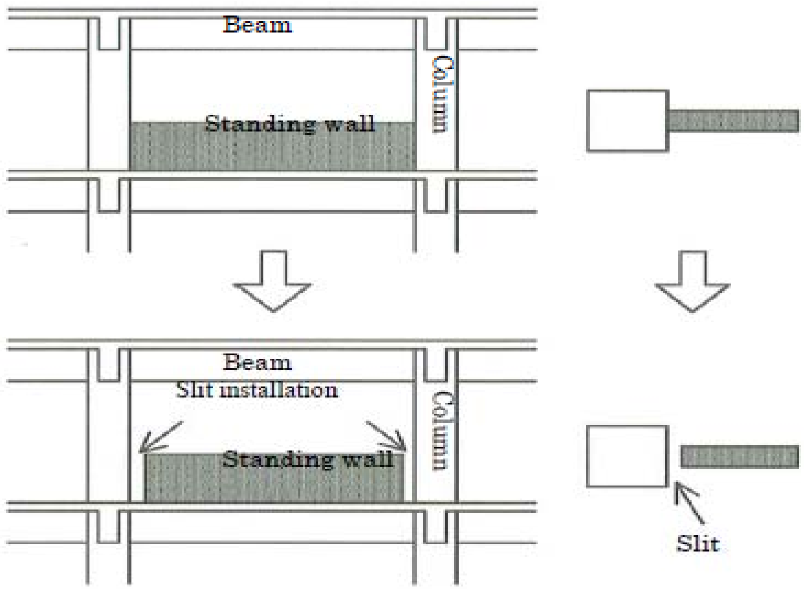
•Construction shall be done by skilled worker since performance of this method is highly dependent construction quality

•Overlap of carbon fiber sheet shall be long enough to ensure the rupture of the material.

Kensetsu Kaikan Building

2. Providing New Seismic Slit

Slits (open joint) are provided between columns and attached standing walls or wing walls



Retrofitting with Structural Slit

Main features of Structural slit:

•Structural slit may be provided in brick wall or RC wall adjacent to column.

•Improve ductility by avoiding short column.

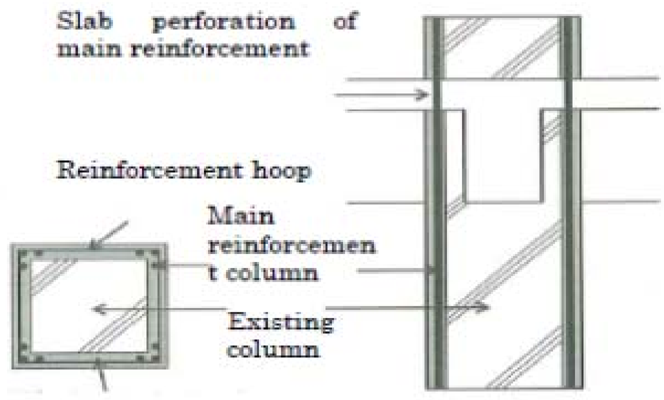
•Secure safety against out of plane behavior of wall to be cut.

•Secure water proofing performance.

3: Concrete Jacketing around RC Column

Reinforced concrete of a thickness of around 10-15cm is jacketed around

existing building columns



STRENGTH OF COLUMN AFTER RC JACKETING

Ultimate Flexural Strength of Jacketed Column

$M_u = a_t \sigma_y g + a_{t2} \sigma_{y2} g_2 + 0.5 N D_2[1-N/(b_2 D_2 F_{c1})]$

Shear force by flexural strength

Qmu = 2Mu/h

Ultimate Shear Strength of Jacketed Column

$Q_{su} = \varphi [0.053 P_{t2}^{0.23}(F_{c1}+18)/(M/Q\, d_2+0.12)$

$+0.85\, Sqrt(P_w.\sigma_{wy}+ P_{w2}.\sigma_{wy2})+0.1\, N/b_2D_2]0.8\, b_2D_2$

F_{c1} =compressive strength of concrete for existing structure, N/mm^2

p_{t2}=tensile reinforcement ratio of jacketed column in %

Retrofitting with Column Jacketing

Main features of RC column jacketing:

•Cross section of existing column is increased.

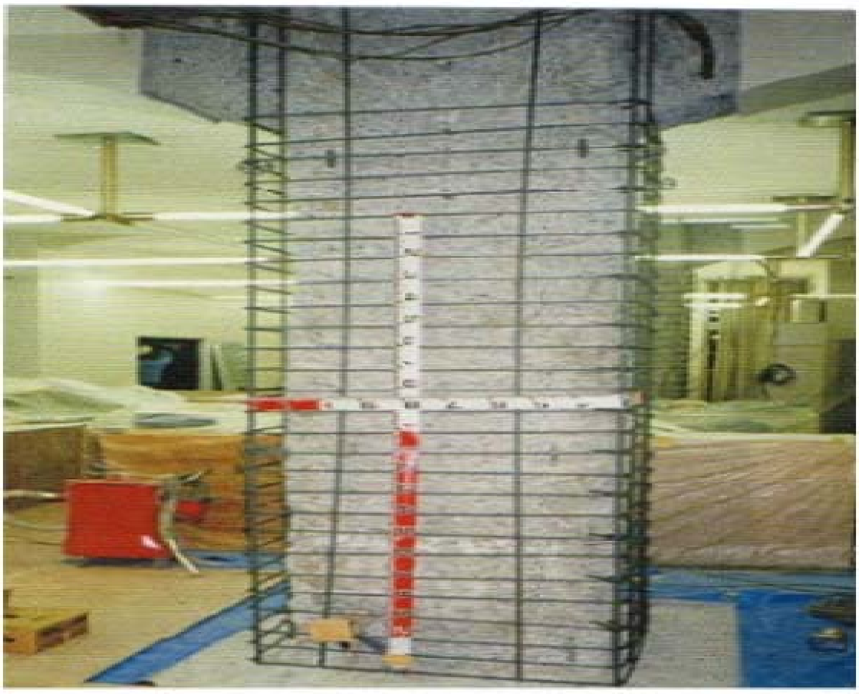
•Usual thickness of jacket is 10 to 15 cm with reinforced concrete.

•Retrofit to improve ductility only.

•Retrofit to improve both ductility and strength.

•In case ductility upgrading provide slit at top and bottom of the column.

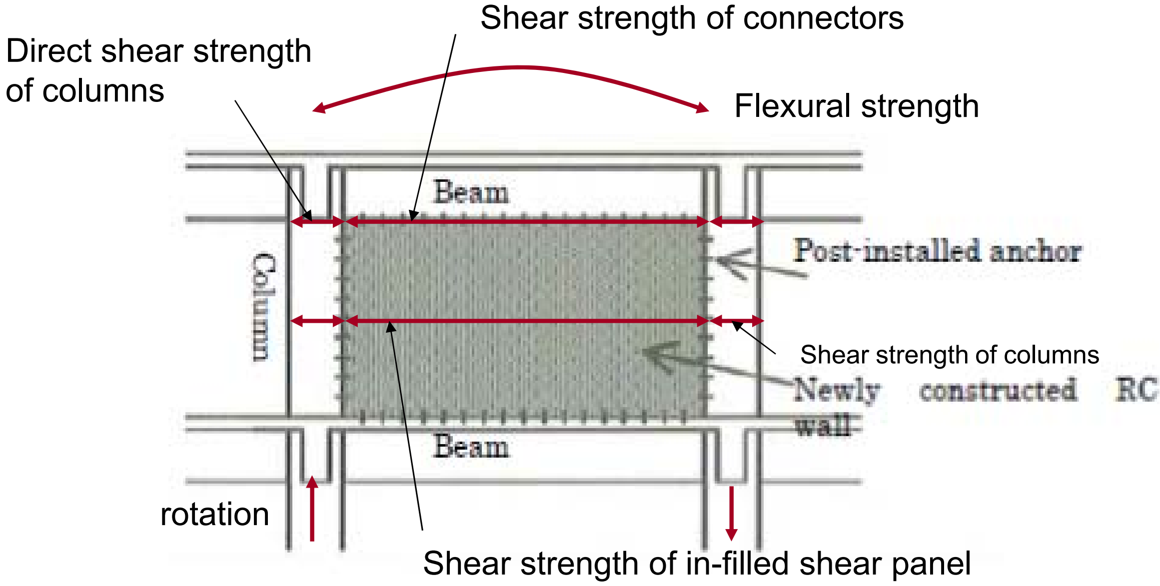
•In case of strength upgrading provide shear key.



Kensetsu Kaikan Building

4: Infilling New RC Shear Wall into Open Frame

Reinforced concrete walls (RC walls) are newly constructed inside existing building column/beam frames



CAPACITY OF INFILLED SHEAR WALL

Shear strength of column:

wQ_{su}=min {wQ'_{su}+ 2∝ $Q_{c,}Q_j$+pQ_c+∝.$Q_{c,}$}

Shear strength of infilled shear panel

wQ'_{su}=max(ρ_w,wσ_y,$\frac{F_{cw}}{20}$+0.5ρ_w.wσ_y).$t_{w.}l'$

Q_c=Smaller value of the other column between

the shear force at the yielding and shear strength.

p_w, $_w\sigma_y$= wall reinforcement ratio and yield strength of wall bar, N/mm^2

F_{cw}= concrete strength of installed wall panels, N/mm2

t_w, l'= wall thickness and clear span of installed wall panel, mm

α= reduction factor, 1 for shear column and 0.7 for flexural column

τ_0

Q_j=Sum of the shear strengths of connectors

underneath the beam

pQ_c=Direct shear strength of column

$=K_{min}.\tau_0.\ b_e.D$

K_{min}= 0.34/(.52+a/D)

b_e=effective width of columns, D=depth of columns,

τ_0= f(σ, F_{c1})

$\sigma = p_g.\ \sigma_y + \sigma_0$

p_g= ratio of a_g to b_e.D

σ_y=yield strength of longitudinal bars of a column

$\sigma_0 = N/b_e.D$

Ultimate flexural strength

$$wM_u = a_t.\ \sigma_{sy}.l_w + 0.5\sum(a_{wy}.\sigma_{wy}).l_w + 0.5Nl_w$$

a_t, $\sum a_{wy}$ = cross sectional area of main bars of a boundary column and Vertical bars in the wall, respectively in mm^2

σ_{sy}, σ_{wy}= yield strength of longitudinal bars of a boundary column and Vertical bars in the wall, respectively (N/mm^2)

N= total axial force in the boundary columns

l_w= distance between the centre of the boundary columns of the wall, mm

Retrofitting with Shear Wall

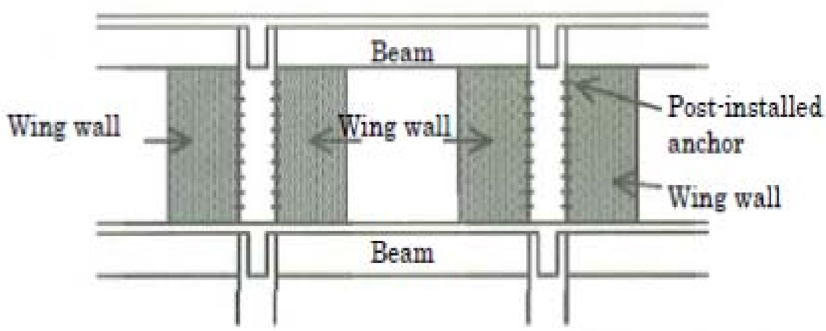
Main features of Shear wall:

- This method is done for strength upgrading.
- Uplift strength of wall shall not be less than shear strength.
- Check structural balance i.e. eccentricity.
- Check capacity of solid shear wall as well as connections with boundary frame.
- Thickness of wall shall not be less than 15 cm but not more than the width of the beam.
- Reduce lighting and ventilation or subdivide inner spaces.

Meguro-ward Government Office

5: Constructing New RC Wing Wall to RC Column

Wing walls of reinforced concrete constructions are newly established in existing building columns



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CAPACITY AFTER ADDING WING WALL:

Ultimate Flexural Strength

$$M_u=(0.9+\beta)a_t\sigma_y D+0.5ND\{1+2\beta-\frac{N}{\alpha_e b D F_{ct}}(\frac{a_t\sigma_y}{N}+1)^2\}$$

Ultimate Shear Strength

$$Q_{su}=\phi\{\frac{0.053\rho_{te}^{0.23}(F_c+18)}{\frac{M}{Q.d_e}+0.12}+0.85\sqrt{\rho_{we}\sigma_{wy}}+0.1\sigma_{0e}\}b_e j_e$$

$\alpha_e=(1+2\alpha\beta)/(1+2\beta)$

F_{c1} =compressive strength of concrete for wing wall , N/mm^2

N= axial force of column, N

b= width of column, D=depth of column,

a_t= gross sectional area of main bars of column in tensile side, mm^2

σ_y= yield strength of main bars of column, N/mm^2 and $\varphi=0.8$

F_c =compressive strength of concrete for existing structure, N/mm^2

$p_{te}= 100a_t/(b_e.d_e)$ $b_e=\alpha_e. b$ in mm

$p_{we}. \sigma_{wy}= p_w, \sigma_{wy}(b/b_e)+ p_{sh}, \sigma_{sy}(t/b_e)$

p_w, σ_{wy}= hoop ratio and yield strength of existing column, N/mm^2
p_{sh}, σ_{sy}=lateral reinforcement ratio of installed wing wall and its yield strength ,N/mm^2
$\sigma_{oe}= N/b_e. j_e$ and $j_e= 7d_e/8$ in mm

Retrofitting with Wing Wall

Main features of Wing wall:

•This method is usually done for strength upgrading.

•Seismic performance may be upgraded by changing failure mechanism from column yielding to beam yielding

.

•Not suitable for column with short span beam, ensure clear span/depth ratio is more than 4.

•Check structural balance i.e. eccentricity.

•Thickness of wall shall not be less than 20 cm.

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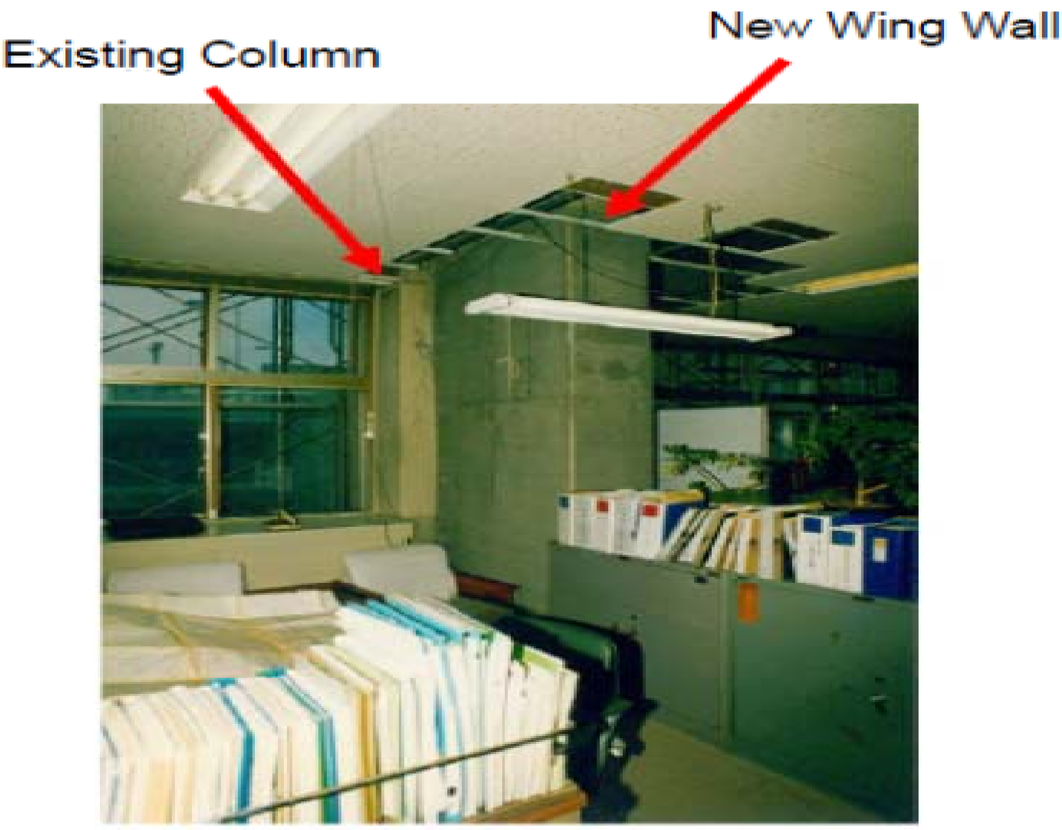

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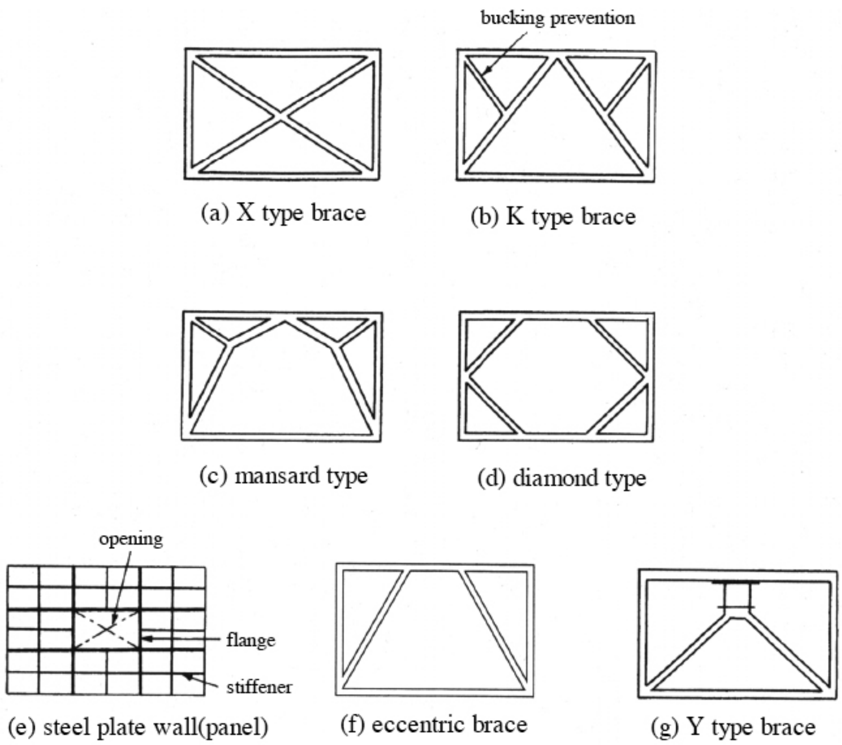
Retrofitting with Steel Frame



Steel bracing:

•Steel member

•Connection

•Headed stud

•Post installed anchor

F = 1.5 to 2.0 subject to failure mode of steel bracing , connection and RC frame

Retrofitting with Steel Frame

Main features of Steel frame:

•Steel framed braced/panel or non-frame brace/panel is inserted into existing RC frame.

•Resistance mechanism after retrofitting may be-(1) strength dominant type, (2) ductility dominant type (3) strength and ductility dominant type.

•Check structural balance i.e. eccentricity.

•Check local buckling of steel member.

•Check capacity of post installed anchor and studs.

•Lighting and ventilation is not so disturbed.

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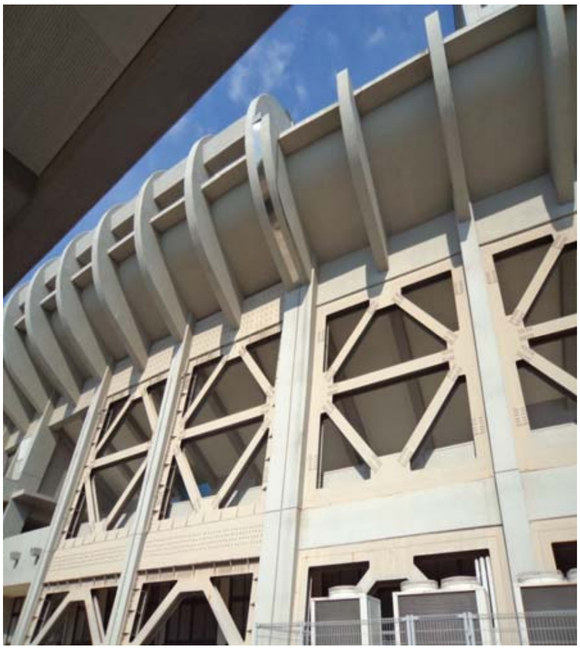
A school building of Japan



A stadium building of Japan

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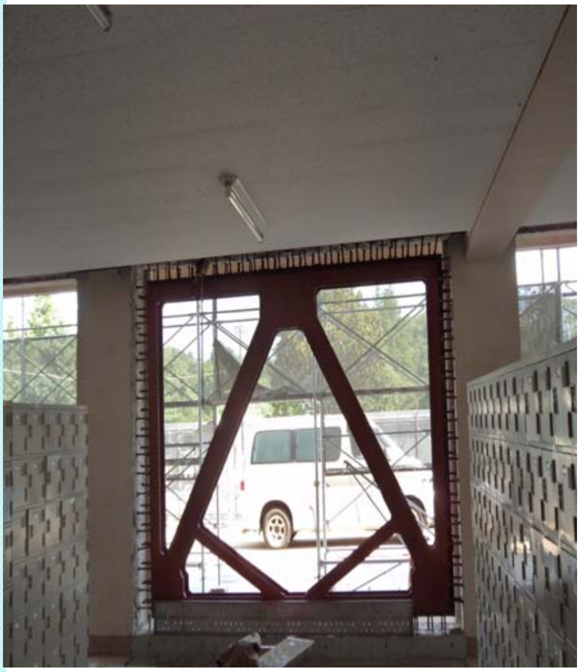

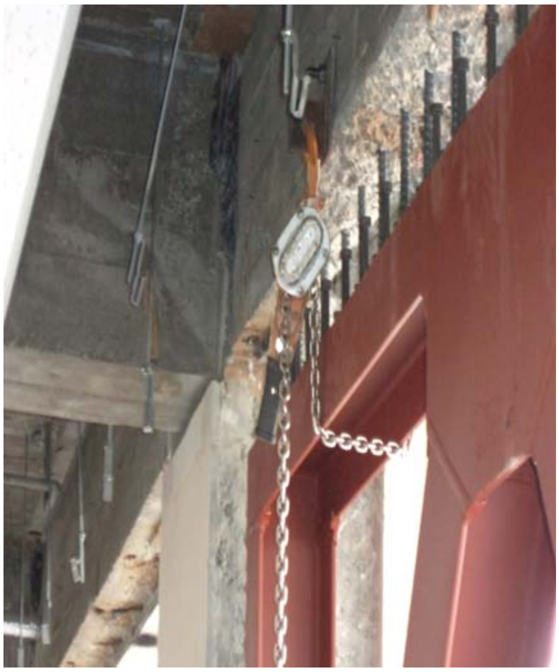

Retrofitting of a school building in Japan

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An Example of Seismic Evaluation and Retrofitting of Existing RC Buildings

2nd level seismic screening is applied, assuming column collapse.

General View of an Essential Hospital in Algiers, Algeria

Source: lecture from Akira INOUE, JICA Expert Team delivered on 09/June/2011

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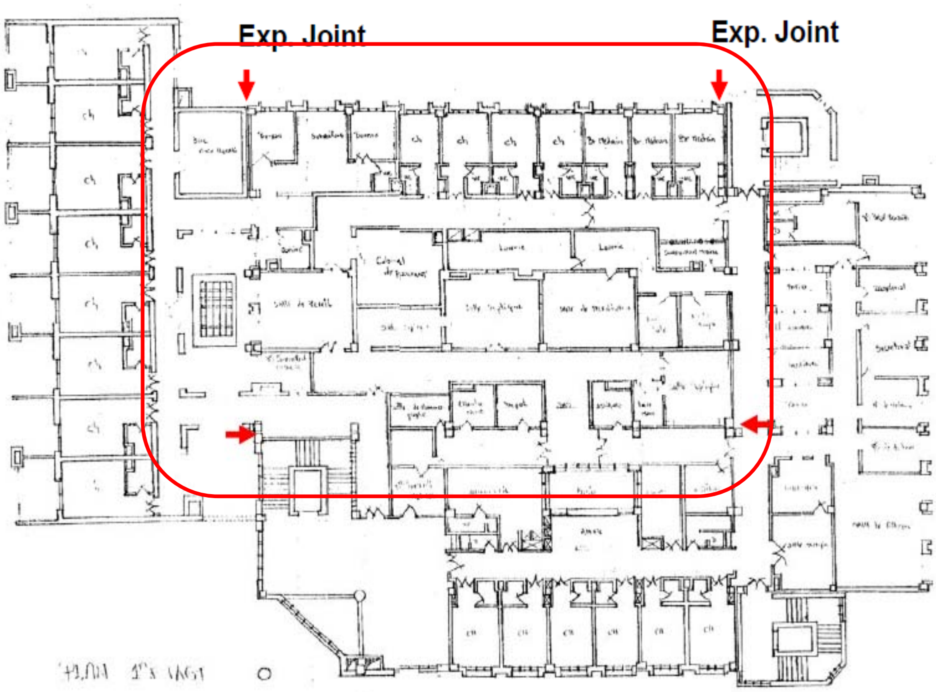
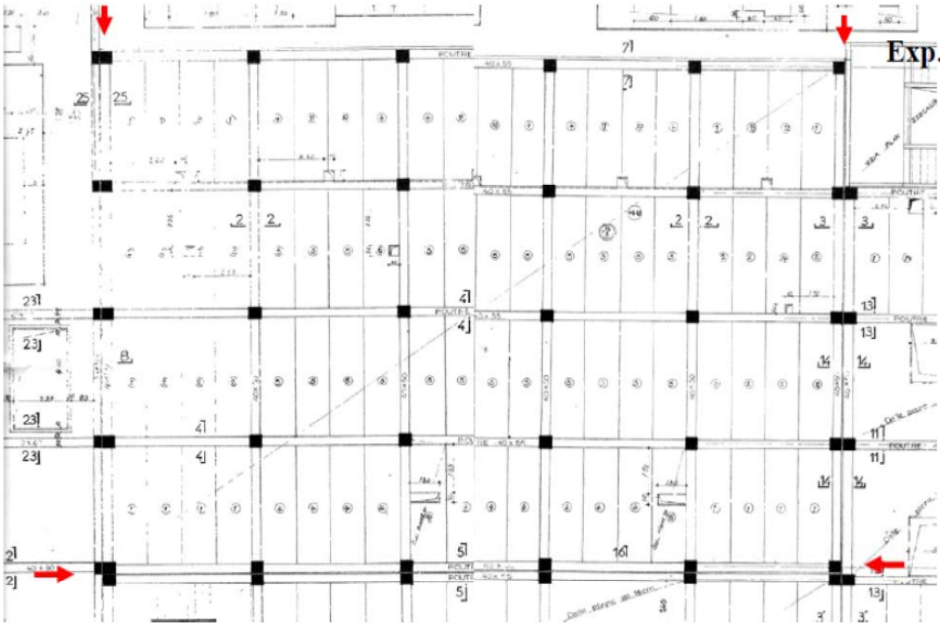
Architectural Plan (Ground floor)



Structural Framing Plan (1st Floor)

One Block of Moment Frame Building is Selected for Seismic Evaluation and Retrofitting



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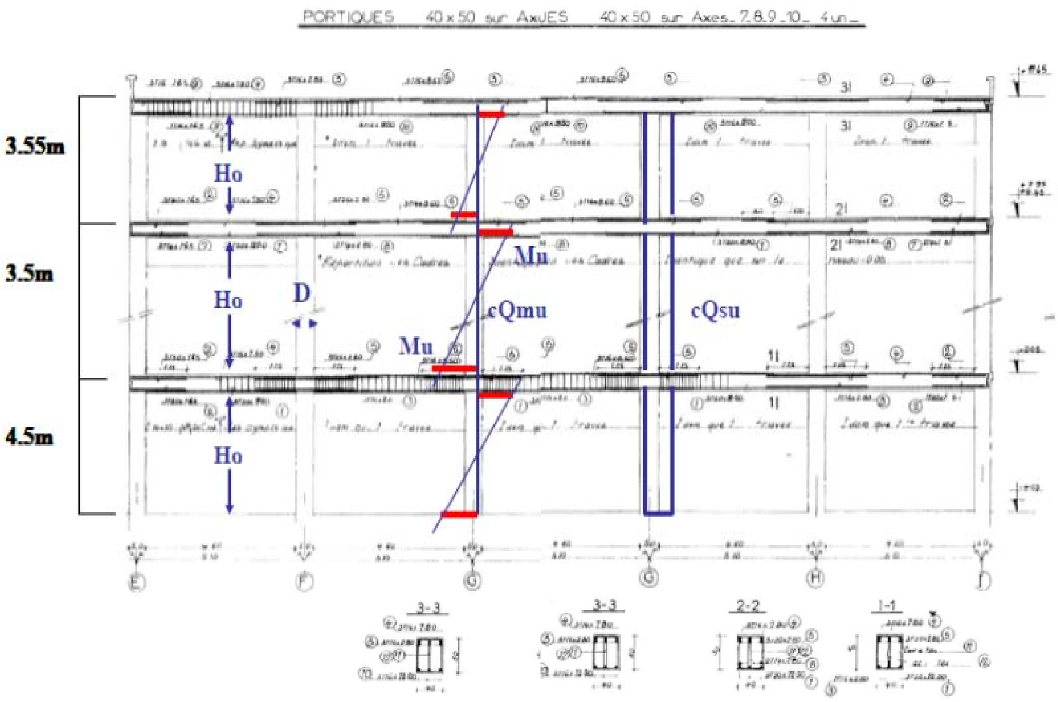
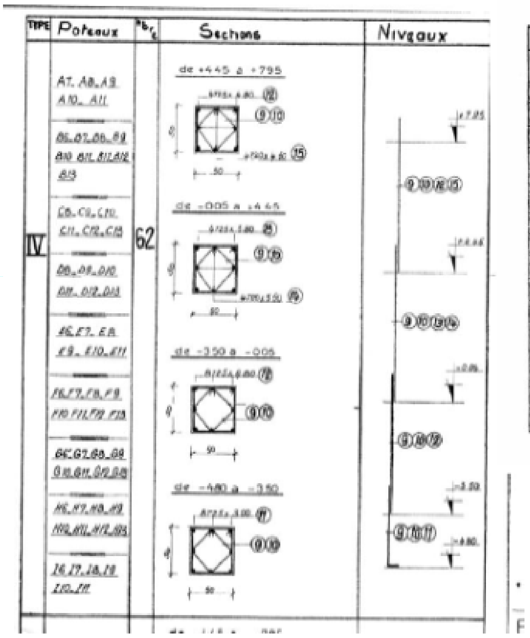
Structural Framing Elevation and Beam Section- design drawing

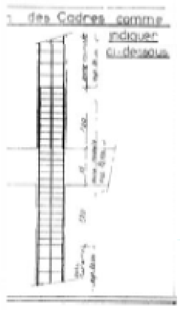


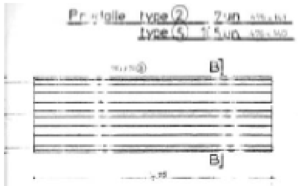
Column Section and Floor Slab System- design drawing



NOMENCLATUR DES ACIERS

Rep	Ø	Type	Nombre		Total	Longueur de barre	Longueur totale	Façonnage
1		16	8	8	64	2 50	160 00	
2		8	80	1	80	1 55	47 00	
3		8	30	1	30	1 15	35 00	
4		14	8	8	64	3 50	240 00	
5		16	-	-	30	2 20	66 00	
6		8	160	8	1280	1 25	1600 00	
7		14	8	7	56	4 30	241 00	
8		16	8	7	56	5 30	297 00	
9		10	-	73	5840	1 35	7884 00	
10		10	-	73	5840	1 96	11388 00	
11		25	8	73	584	3 00	1752 00	
12		25	-	73	888	4 80	4265	
13		25	-	72	312	5 80	1810 00	
14		20	4	66	264	5 50	1452 00	
15		20	-	72	304	4 50	1368 00	
16		25	8	1	8	9 30	75 00	





Building Dimensions, Weight of Building and Materials

- **Building Dimensions**
- X direction 6.0m span x 5=30m, Y direction 5.1m x 4=20.4m (grid line)
- Storey Height GF 4.5m, 1F 3.5m, 2F 3.55m total 11.55m
- Clear Length of Column Y direction 1F 4.0m, 2F 3.0m, 3F 3.05m

- **Unit Weight per Floor Area (Supposed Condition)**
- Roof 11 kN/m^2 (1.12tf/m^2)
- 1st Floor, 2nd Floor 14 kN/m^2 (1.43tf/m^2)

- **Weight of Building**
- Roof 7012kN
- 2nd Floor 8924kN 15936kN (Roof + 3rd floor)
- 1st Floor 8924kN 24860kN (Roof + 3rd + 2nd floor)

- **Material** (from Design Drawings)
- Re-bar Main Bars High Strength 412N/mm^2 (4200kg/cm^2) $\Phi \leqq 20$nn
- 392N/mm^2 (4000kg/cm^2) $\Phi > 20$nn
- Hoops, Stirrups Mild Steel 235N/mm^2 (2400kg/cm^2)
- Concrete (28 days strength) 27N/mm2 (275kg/cm^2)

Strength Index C,
Flexural Strength of Columns

For $N_{max} \geq N > 0.4b \cdot D \cdot F_c$

$$M_u = \left\{0.8a_t \cdot \sigma_y \cdot D + 0.12b \cdot D^2 \cdot F_c\right\} \cdot \left(\frac{N_{max} - N}{N_{max} - 0.4b \cdot D \cdot F_c}\right)$$

For $0.4b \cdot D \cdot F_c \geq N > 0$

(N·mm)

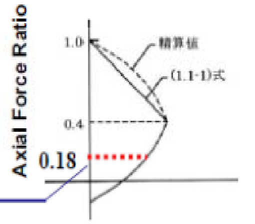
$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.5N \cdot D \cdot \left(1 - \frac{N}{b \cdot D \cdot F_c}\right)$$

GF column=0.18

For $0 > N \geq N_{min}$

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.4N \cdot D$$

(A1.1-1)



Flexural Strength of Column

where:

N_{max} = Axial compressive strength $= b \cdot D \cdot F_c + a_g \cdot \sigma_y$ (N).

N_{min} = Axial tensile strength $= -a_g \cdot \sigma_y$ (N).

N = Axial force (N).

a_t = Total cross sectional area of tensile reinforcing bars (mm²).

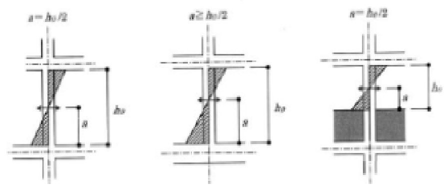
a_g = Total cross sectional area of reinforcing bars (mm²).

b = Column width (mm).

D = Column depth (mm).

σ_y = Yield strength of reinforcing bars (N/mm²).

F_c = Compressive strength of concrete (N/mm²).



Clear Height of Column

(b) The multi layered reinforcement shall be considered in using Eq. (A1.1-1).

(c) In calculating the ultimate flexural strength of columns, another calculation method su as based on rigid-plastic theory may be used instead.

Shear Strength of Columns

$$Q_{su} = \left\{\frac{0.053p_t^{0.23}(18 + F_c)}{M/(Q \cdot d) + 0.12} + 0.85\sqrt{p_w \cdot {}_s\sigma_{wy}} + 0.1\sigma_0\right\} \cdot b \cdot j \quad \text{(N)} \qquad \text{(A1.1-2)}$$

where:

p_t = Tensile reinforcement ratio (%).

p_w = Shear reinforcement ratio, p_w =0.012 for $p_w \geq 0.012$.

${}_s\sigma_{wy}$ = Yield strength of shear reinforcing bars (N/mm²).

σ_0 = Axial stress in column (N/mm²).

d = Effective depth of column. D-50mm may be applied.

$\frac{M}{Q}$ = Shear span length. Default value is $\frac{h_0}{2}$.

h_0 = Clear height of the column.

j = Distance between centroids of tension and compression forces, default value is $0.8D$.

(b) If the value of $M/(Q \cdot d)$ is less than unity or greater than 3, the value of $M/(Q \cdot d)$ shall be unity or 3 respectively in using Eq. (A1.1-2). And if the value of σ_0 is greater than 8N/mm², the value of σ_0 shall be 8N/mm² in using Eq. (A1.1-2).

Flexural Strength of Column and Margin against Shear Failure

	Internal Columns				External Columns			
	Mu(kN･m)	cQmu(kN)	cQsn(kN)	cQsn/cQmu	Mu(kN･m)	cQmu(kN)	cQsn(kN)	cQmu/cQmu
2F	286	191	349	1.83	250	167	337	2.02
1F	375	250	384	1.53	303	202	356	1.76
GF	451	226	418	1.85	352	176	374	2.13

Mu: Ultimate Flexural Strength of Column (A1.1-1)

cQmu: Shear Force at the Ultimate Flexural Strength of Column $cQmu=Mu/(h_0/2)$

cQsn: Ultimate Shear Strength of Column (A1.1-2)

cQsn/cQmu: Strength Margin for Shear Failure of Flexural Column

Strength Index (C)

	ΣW(kN)	Internal Columns 12nos		External Columns 18nos		Total
		cQmux12	C=cQmux12/ΣW	cQmux18	C=cQmux18/ΣW	C
2F	7012	2288	0.326	3001	0.428	0.754
1F	15936	3002	0.188	3635	0.228	0.416
GF	24860	2707	0.109	3165	0.127	0.236

ΣW: Total Weight (Dead Load plus Live Load) Supported by the Storey Concerned

C: Strength Index C=Qu/ΣW (12)

Qu: Ultimate Lateral Load-carrying Capacity of the Vertical Members in the Storey Concerned, (in this case, Qu=cQmu(internal)x12+cQmu(external)x18)

Ductility Index (F) , Yield Deflection of Flexural Column

$R_{my} = (h_0 / H_0) \cdot {}_cR_{my} \geq R_{250}$ (A1.3-1)

where, $h_0 / H_0 \leq 1.0$ → $h_0/H_0=1.0$

→ ${}_cR_{my} = {}_cR_{150}$ for $h_0 / D \geq 3.0$ → cRmy=cR_{150}=1/150

${}_cR_{my} = {}_cR_{250}$ for $h_0 / D \leq 2.0$ (A1.3-2)

${}_cR_{my}$ is set by interporation for $2.0 < h_0 / D < 3.0$

where:

h_0 = Clear height of the column.

H_0 = Standard clear height of the column from the bottom of the upper floor beam to the top of the lower floor slab.

D = Column depth.

${}_cR_{150}$ = Standard drift angle of the column (measured in the clear height of column), 1/150.

${}_cR_{250}$ = Standard drift angle of the column (measured in the clear height of column), 1/250.

R_{250} = Standard inter-story drift angle, 1/250.

${}_cR_{my}$ = Yield drift angle of the column (measured in the clear height of column).

The value of ${}_cR_{my}$ shall not be greater than that of ${}_cR_{max}$ specified in the section 1.2(3) of Supplementary Provisions.

Plastic Drift Angle of Column

${}_cR_{mp} = 10({}_cQ_{su} / {}_cQ_{mu} - q) \cdot {}_cR_{my} \geq 0$ (A1.2-3) → cQsu/cQmu(Margin for Shear Failure)=1.83 to 2.13

→ $q = 1.0$ for $s \leq 100mm$

$q = 1.1$ for $s > 100mm$ (A1.2-4) → cRmp=0.055(1/18) to 0.075(1/13)

where:

${}_cQ_{su}$ = Ultimate shear strength of the column, calculated with Eq. (A1.1-2) in principle.

${}_cQ_{mu}$ = Shear force at the ultimate flexural strength of the column. The largest moment capacity shall be used under the working axial force, in case axial force of column is greater than the balanced axial force.

s = Spacing of hoops.

Storey Drift Angle at Ultimate Flexural Strength of Column

$$R_{mu} = (h_0 / H_0) \cdot {}_cR_{mu} \geq R_{250} \qquad \text{(A1.2-1)}$$

where, $h_0 / H_0 \leq 1.0$ ⟶ **h_0/H_0=1.0, Rmu=cRmu**

$${}_cR_{mu} = {}_cR_{my} + {}_cR_{mp} \leq {}_cR_{30} \qquad \text{(A1.2-2)}$$

⟶ **cRmu=cRmy(1/150)+cRmp(A1.2-3)**

where:

h_0 = Clear height of column.

H_0 = Standard clear height of column from bottom of the upper floor beam to top of the lower floor slab.

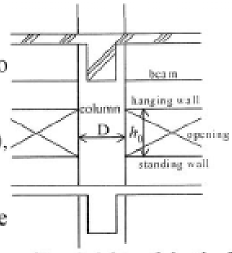
${}_cR_{my}$ = Yield drift angle of column (measured in clear height of column), specified in the section 1.3 of Supplementary Provisions.

${}_cR_{mu}$ = Drift angle at the ultimate flexural strength of column (measured in the clear height of column).

${}_cR_{mp}$ = Plastic drift angle of the column (measured in the clear height of column), specified in the section 1.2(2) of Supplementary Provisions.

${}_cR_{30}$ = Standard drift angle of the column (measured in the clear height of column), 1/30.

R_{250} = Standard inter-story drift angle, 1/250.



Clear height and depth of column

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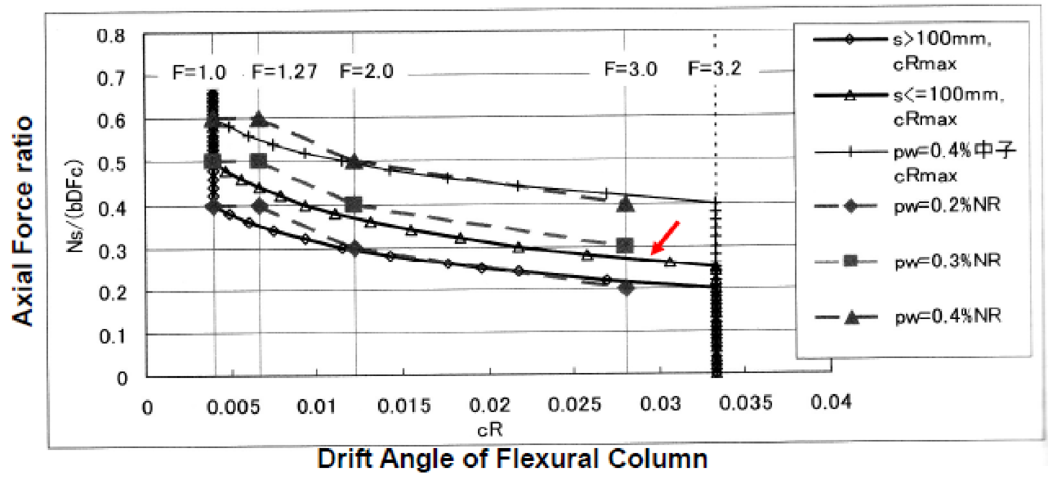
Upper Limit of Drift Angle of Column

- **(A1.2-5)**
- **1. Axial Force Ratio (A1.2-6)**
- **N/bDFc=0.18＜0.25 ----- cR30**
- **2. Shear Force (A1.2-7)**
- **cτu/Fc=0.042＜ 0.2 -----cR30**
- **3. Tensile Reinforcement Ratio (A1.2-8)**
- **Pt=0.52%＜1.0% ----cR30**
- **4. Spacing of Hoops (A1.2-8)**
- **S/db=@100/Φ20(Main Bar)=5＜8 ----cR30**
- **5. Clear Height of Column (A1.2-9)**
- **H0/D=3.0m(2F)/0.5m=6≧2 ----cR30**
- **Where, cR30: Standard Drift Angle of Column (measured in the**
- **clear height of column), 1/30**
- ⟶ **Ductility Factor μ=5.0 (upper limit is used)**

Upper Limit of Drift Angle of Flexural Column and Axial Force Ratio



Ductility Index (F) of Flexural Column

(i) In case $R_{mu} < R_y$ (Rmu)

$$F = 1.0 + 0.27\frac{R_{mu} - R_{250}}{R_y - R_{250}} \qquad (15)$$

(ii) In case $R_{mu} \geq R_y$ (Rmu)

$$F = \frac{\sqrt{2R_{mu}/R_y - 1}}{0.75 \cdot (1 + 0.05R_{mu}/R_y)} \leq 3.2 \qquad (16)$$

→ Rmu/Ry=5.0, F=3.2

where:

R_y = Yield deformation in terms of inter-story drift angle, which in principle shall be taken as R_y=1/150.

R_{250} = Standard inter-story drift angle (corresponding to the ductility index of the shear wall), R_{250} = 1/250.

R_{mu} = Inter-story drift angle at the ultimate deformation capacity flexural failure of the column member, calculated by Eq. (A1.2 in the Supplementary Provisions 1.2(1).

Ductility-dominant Basic Seismic Index of Structure Eo

$$E_0 = \frac{n+1}{n+i}\sqrt{E_1^2 + E_2^2 + E_3^2}$$ **Eo=(3+1)/(3+1)・C1・F1 (GF)** (4)

where:

E_1 = $C_1 \cdot F_1$.

E_2 = $C_2 \cdot F_2$.

E_3 = $C_3 \cdot F_3$.

C_1 = The strength index C of the first group (with small F index).

C_2 = The strength index C of the second group (with medium F index).

C_3 = The strength index C of the third group (with large F index).

F_1 = The ductility index F of the first group.

F_2 = The ductility index F of the second group.

F_3 = The ductility index F of the third group.

Strength-dominant Basic Seismic Index of Structure Eo

$$E_0 = \frac{n+1}{n+i}\left(C_1 + \sum_j \alpha_j C_j\right) \cdot F_1 \quad (5)$$

where:

α_j = Effective strength factor in the j-th group at the ultimate deformation R_1 corresponding to the first group (ductility index of F_1), given in Table 3.

Table 3 Effective strength factor

Cumulative point of the first group F_1 =0.8 (Drift angle R_1=R_{500}=1/500)		
	F_1	F_1=0.8
	R_1	R_1=R_{500}
Second and higher groups	Shear (R_{su}=R_{250})	α_S
	Shear (R_{250}< R_{su})	α_S
	Flexural (R_{my}=R_{250})	0.65
	Flexural (R_{250}<R_{my}<R_{150})	α_m
	Flexural (R_{my}=R_{150})	0.51
	Flexural and shear walls	0.65

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Irregularity Index S_D Table 6

Expansion Joint 4cm/1150cm(height)=1/287.5<1/200, 1/100 q2d=0.95

Storey Height Uniformity i=3.5m(1F)/4.5m(GF)=0.78<0.8 q2i=0.975 S_D=0.926

Table 6 Classification of items and G, R-values

			Gi (Grade)			*R* (adjustment fator)	
			1.0	0.9	0.8	*R1i*	*R2i*
Horizontal balance	a	Regularity	Regular a1	Nearly regular a2	Irregular a3	1.0	0.5
	b	Aspect ratio of plan	b≦5	5<b≦8	8<b	0.5	0.25
	c	Narrow part	0.8≦c	0.5≦c<0.8	c<0.5	0.5	0.25
	d	Expansion joint *1	1/100≦d	1/200≦d< 1/100	D<1/200	0.5	0.25
	e	Well-style area	e≦0.1	5<e≦8	0.3<e	0.5	0.25
	f	Eccentric well-style area*2	f_1≦0.4 & f_2≦0.1	f_1≦0.4 & 0.1<f_2≦0.3	0.4<f_1 or 0.3<f_2	0.25	0
	g						
Elevation balance	h	Underground floor	1.0≦h	0.5≦h< 1.0	h<0.5	0.5	0.5
	i	Story height uniformity	0.8≦I	0.7≦I<0.8	I<0.7	0.5	0.25
	j	Soft story	No soft story	Soft story	Eccentric soft story	1.0	1.0
	k						
Eccentricity	l	Eccentricity*3	l≦0.1	0.1<*l*≦0.15	0.15<*l*		1.0
	m						1.0
Stiffness	n	(Stiffness/mass)Ratio of above and below stories	n≦1.3	1.3<n≦1.7	1.7<n		1.0
	o						1.0

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Time Index T=0.95 (assumed) Table 8

Table 8 Evaluation of time index by the second level inspection (-story) for the second level screening

Portion	Range / Item, Degree	Structural cracking and deflection a	b	c	Deterioration and aging a	b	c
	Degree	1. Cracking caused by uneven settlement. 2. Shear or inclined cracking in beams, walls, and/or columns, observed evidently.	1. Deflection of a slab and/or beam, affecting on the function of non-structural element. 2. Same as left but not visible from some distance. 3. Same as above but can be observed from some distance.	1. Minute structural cracking not corresponding to the items a or b. 2. Deflection of a slab and/or beam, not corresponding to the item a or b.	1. Cracking by concrete expansion due to the rust of reinforcing bar. 2. Rust of reinforcing bar. 3. Cracking caused by a fire disaster. 4. Deterioration of concrete caused by chemicals.	1. Seep of the rust of reinforcing bar due to rain water or water leak. 2. Neutralization to the depth of reinforcing bar or equivalent aging. 3. Spalling off of finishing materials.	1. Remarkable blemish of concrete due to rain water, water leak, and chemicals. 2. Deterioration or slight spalling off of a finishing material.
I Slab including sub-beam	1) 1/3 or more of total floor	0.017	0.005	0.001	0.017	0.005	0.001
	2) 1/3~1/9	0.006	0.002	0	0.006	0.002	0
	3) 1/9 or less	0.002	0.001	0	0.002	0.001	0
	4) 0 #	0	0	0	0	0	0
II Beam	1) 1/3 or more of total number of members for each direction	0.05	0.015	0.004	0.05	0.015	0.004
	2) 1/3~1/9	0.017	0.005	0.001	0.017	0.005	0.001
	3) 1/9 or less	0.006	0.002	0	0.006	0.002	0
	4) 0 #	0	0	0	0	0	0
III Wall & Column	1) 1/3 or more of total number of members	0.15	0.045	0.011	0.15	0.045	0.011
	2) 1/3~1/9	0.05	0.015	0.004	0.05	0.015	0.004
	3) 1/9 or less	0.017	0.005	0.001	0.017	0.005	0.001
	4) 0 #	0	0	0	0	0	0
Mark-down	Subtotal						
Total	Ground total	P1			P2		

Remark(#). The item 4) may be adopted in case where there are no areas or members with aging defect, and the maintenance condition of the building could be recognized as very good.

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2nd Level Seismic Screening
Seismic Index of Structure, Is, Y Direction

	C	F	n+1/n+i	Eo	S_D	T	Is
2F	0.76	3.2	0.67	1.63	1.11*	0.95	1.72*
1F	0.42	3.2	0.80	1.07	1.11*	0.95	1.13*
GF	0.24	3.2	1.00	0.76	1.00*	0.95	0.72*

C: Strength Index (12), (A1.1-1), (A1.1-2)

F: Ductility Index (16)

n: Number of storey

Eo: Basic Seismic Intensity of Structure Eo=(n+1/n+i)CF (4)

S_D: Irregularity Index (Expansion Joint x0.95, Storey Height Uniformity x0.975, Underground Floor x1.20*, Stiffness/mass Ratio x1.0(3F,2F), 0.9(1F)*) Table 6

T: Time Index (0.95 is used) Table 8

Is: Seismic Index of Structure Is=EoS_DT (1)

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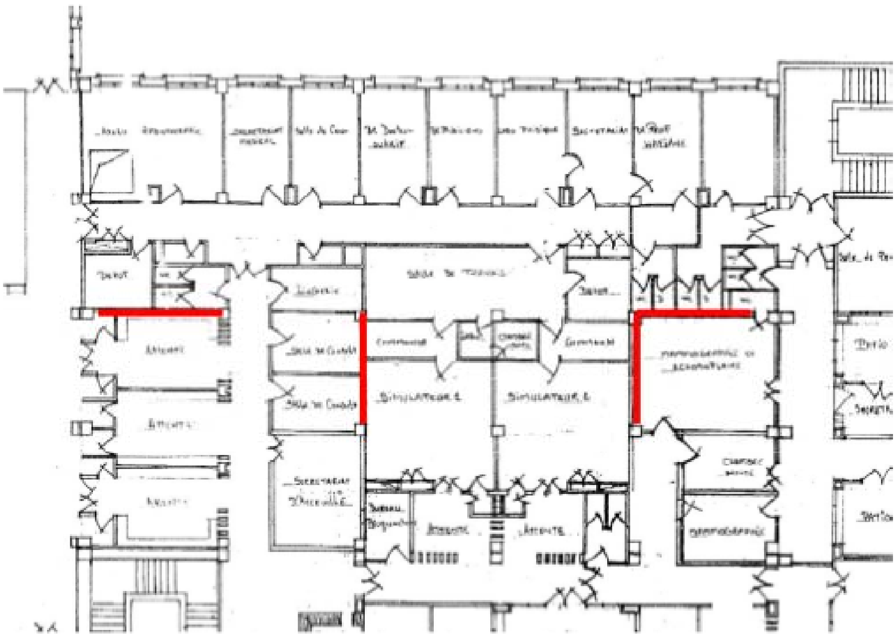
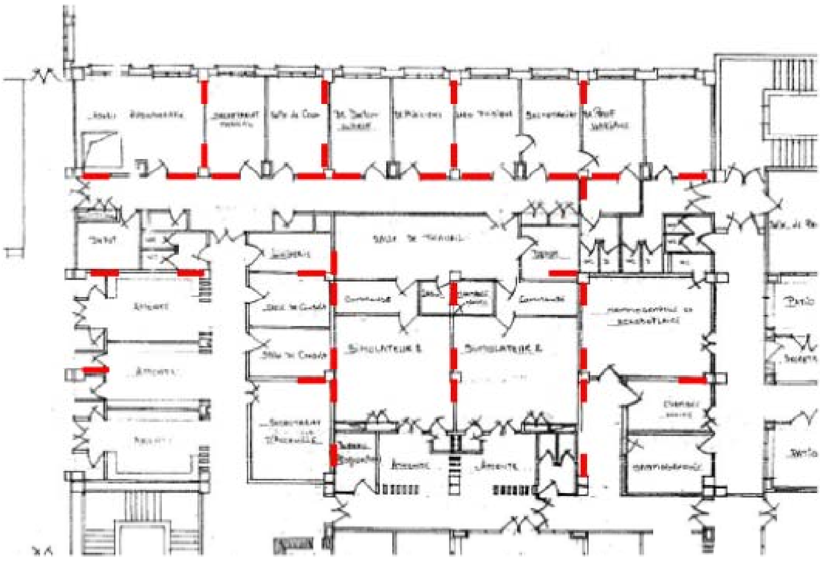
Case 1: Retrofit by RC Walls, 1st Storey



Case 2: Retrofit by Wing-walls, Ground Storey



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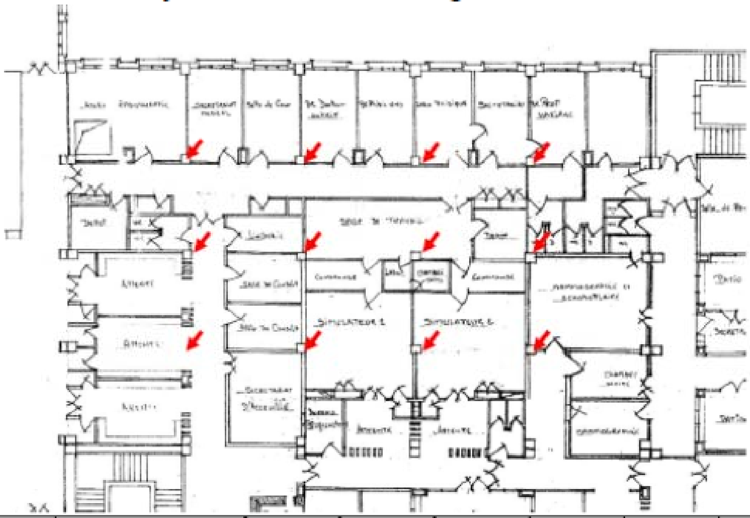
Case 3: Retrofit by Column Jacketing, Ground floor

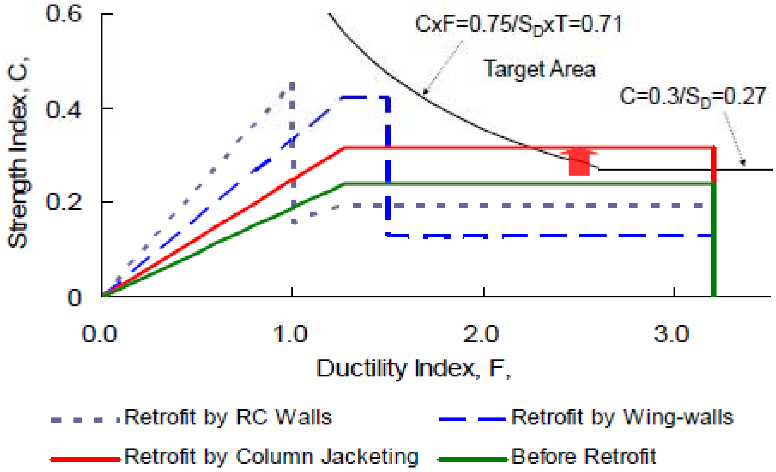


Direction	C		Column	Total C	F	Eo(4)	n+1/n+i	Is	$C_T S_D$
X	0.00708	(perimeter)	18	0.3158	3.2	1.01	1.0	1.07	0.35
	0.0157	(center)	12						
Y	0.00708	(perimeter)	18	0.3134	3.2	1.00	1.0	1.06	0.35
	0.0155	(center)	12						

S_D=1.11, T=0.95 **Strength increase 0.316/0.24=1.32**

Strength Index (*C*) and Ductility Index (*F*)

in X direction of 1st Storey, column jacketing is proposed



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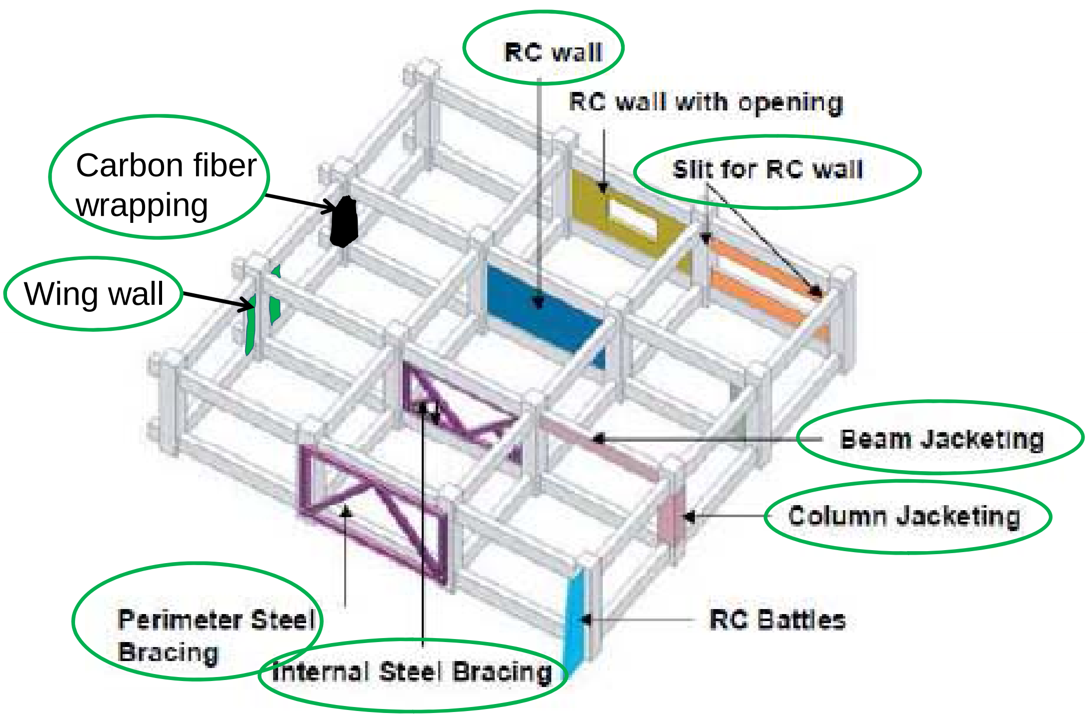
THANK YOU

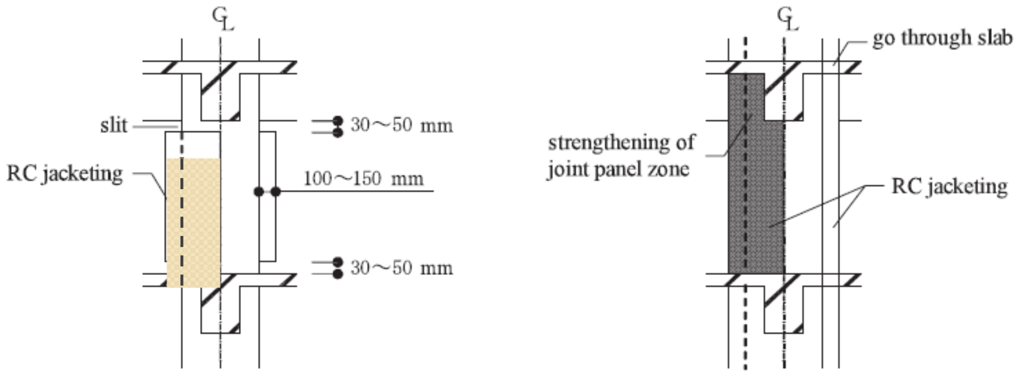
Retrofitting Works

Md. Sohel Rahman
Executive Engineer
PWD Design Division-4
and
Team Leader, Component 3
CNCRP Project

Methods of Retrofitting



Retrofitting with Column Jacketing

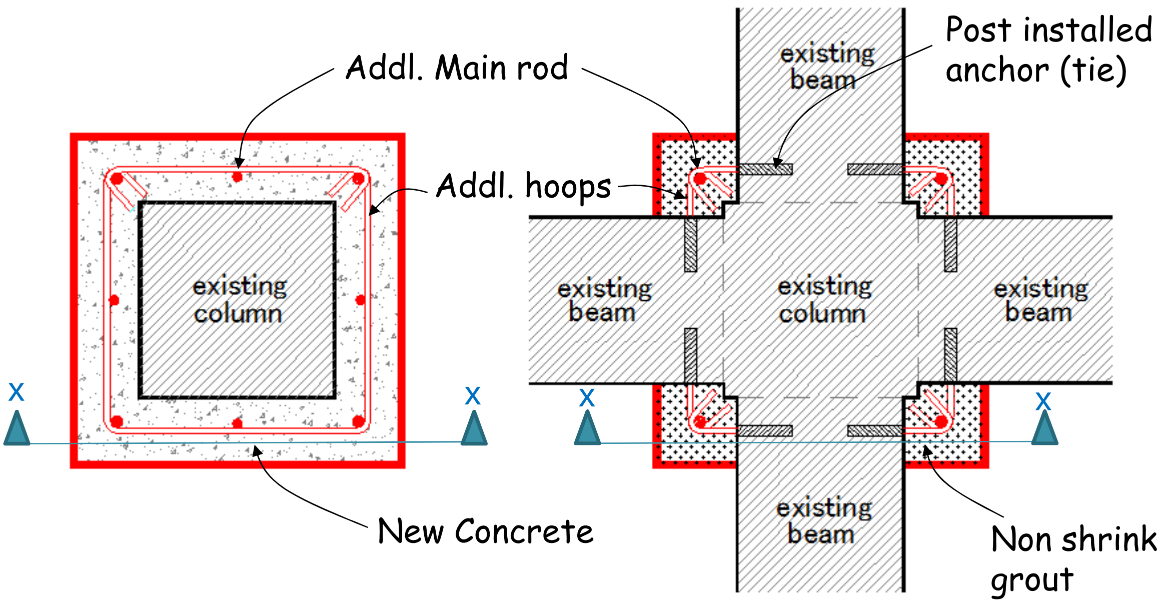


(a) in case of increase in shear strength (b) in case of increase in flexural, shear and axial strength

Figure TN.16 Column strengthening with RC jacketing

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

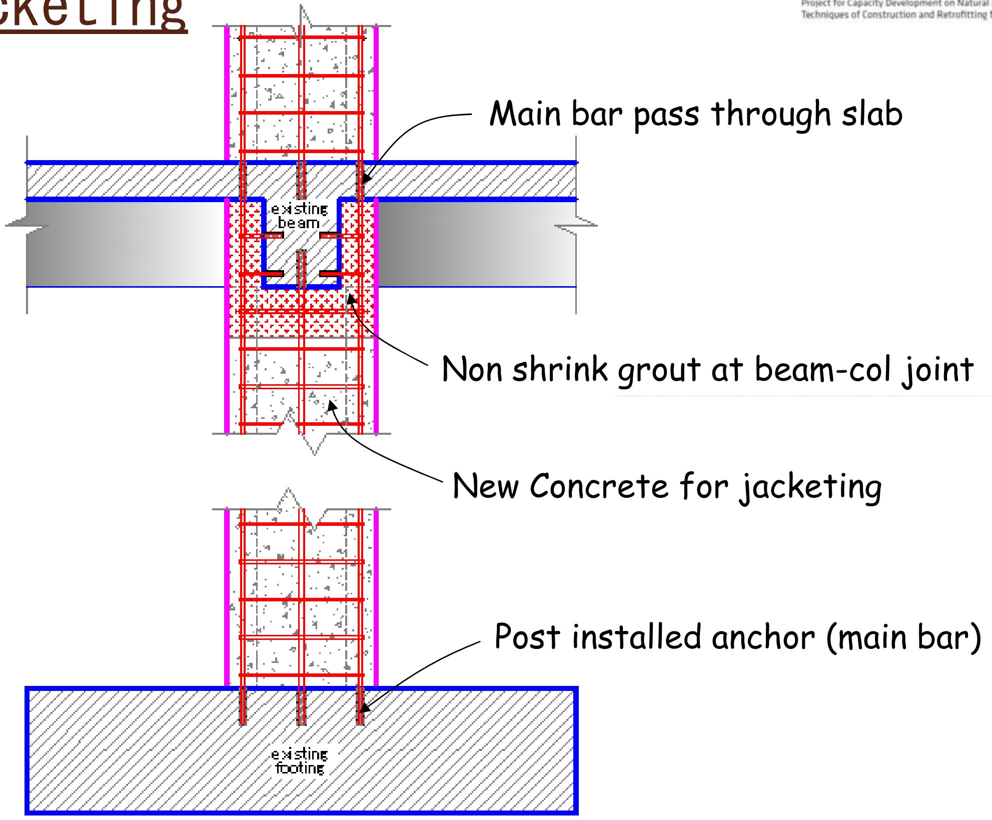
Retrofitting with Column Jacketing



Section of column at mid span

Section of column at beam-col joint

Retrofitting with Column Jacketing



Section of column jacketing

Retrofitting with Column Jacketing

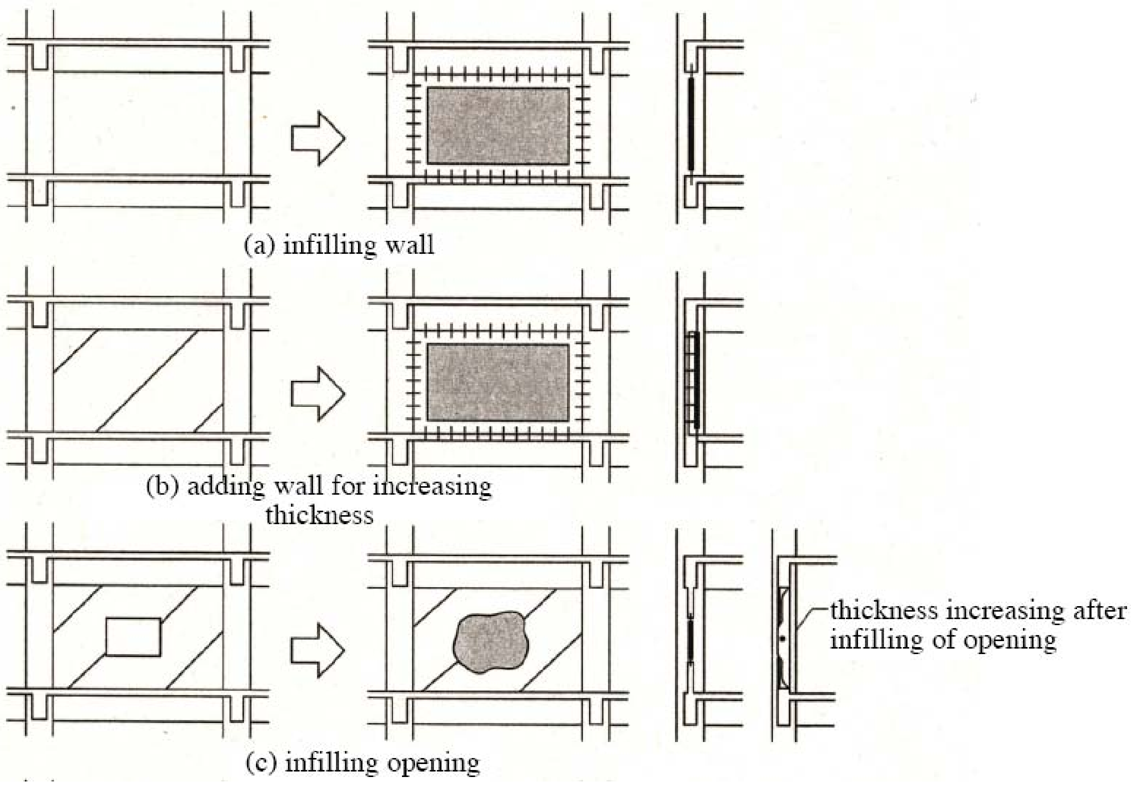
RC Column Jacketing

Test Work of CNCRP in 2012

RC Column Jacketing through beam-col joint

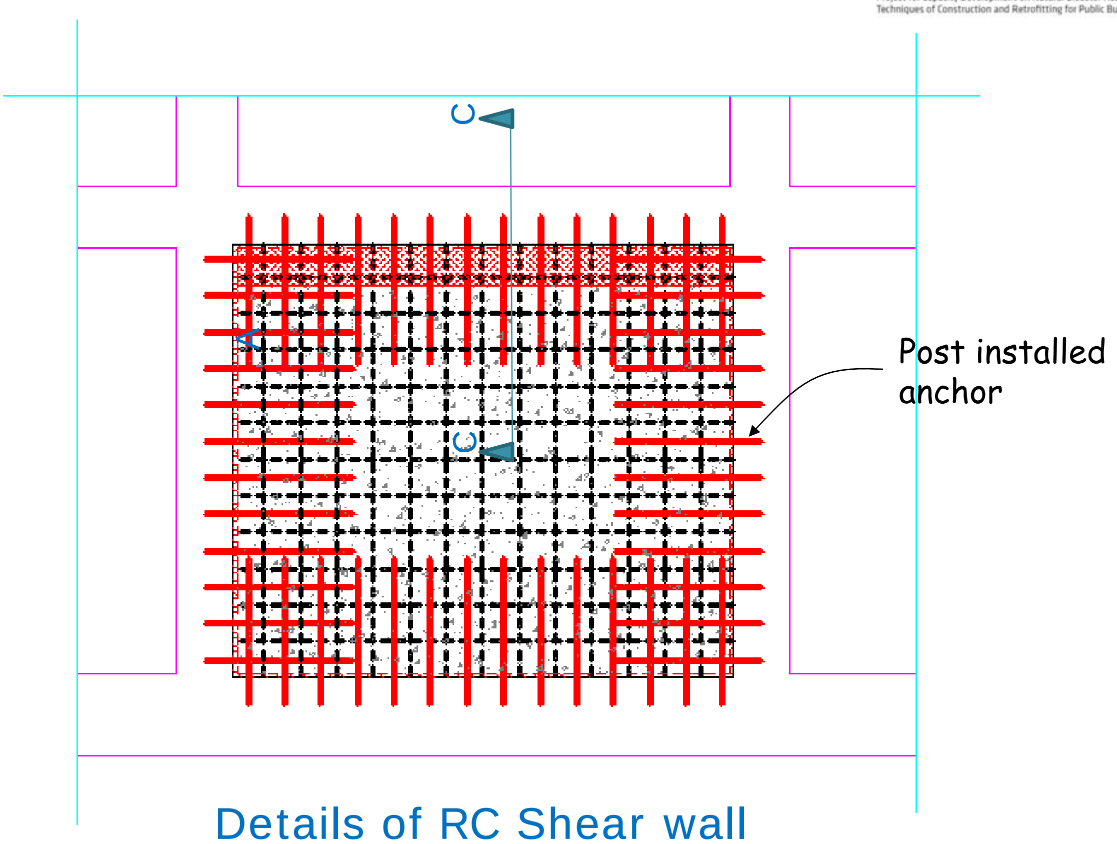
Test Work of CNCRP in 2013

Retrofitting with Shear Wall



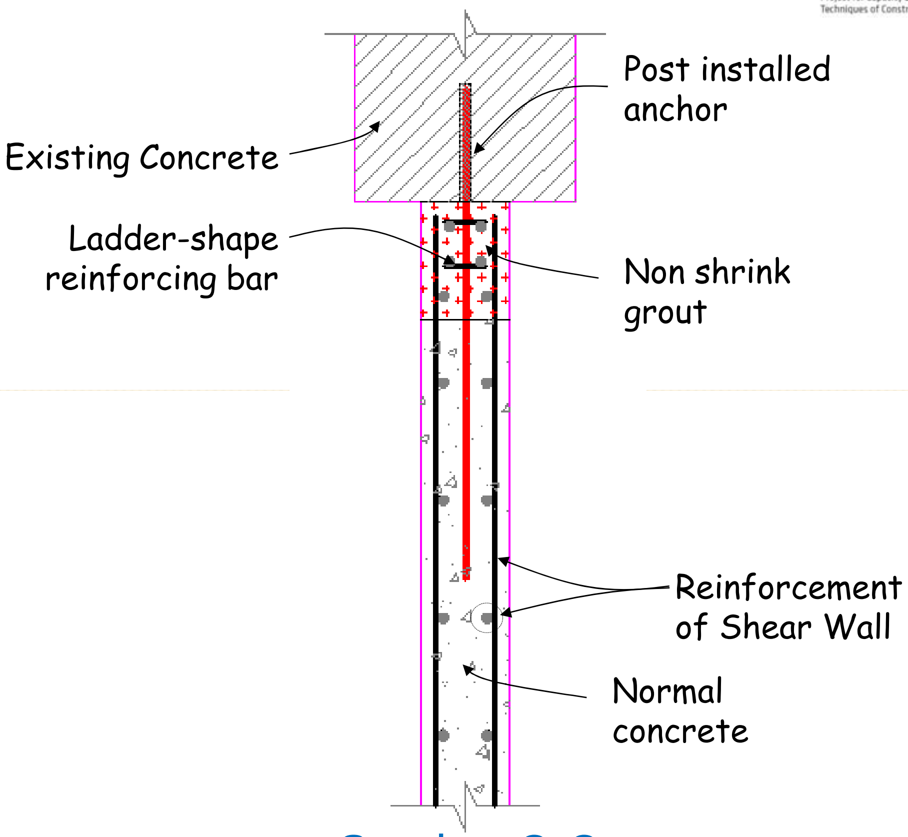
SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Retrofitting with Shear Wall



Details of RC Shear wall

Retrofitting with Shear Wall



Section C-C

Retrofitting with Shear Wall

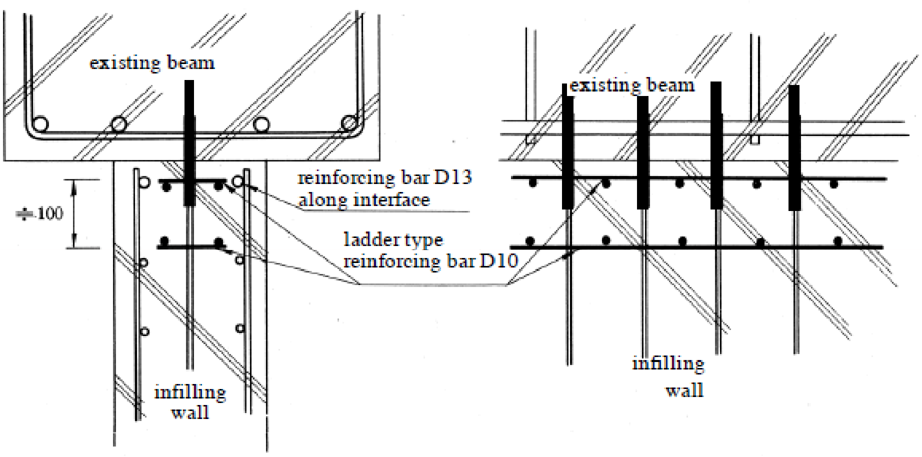


Figure TN.12 Strengthening against splitting with ladder type reinforcing bars
(quoted from the figure on page 98 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Retrofitting with Shear Wall

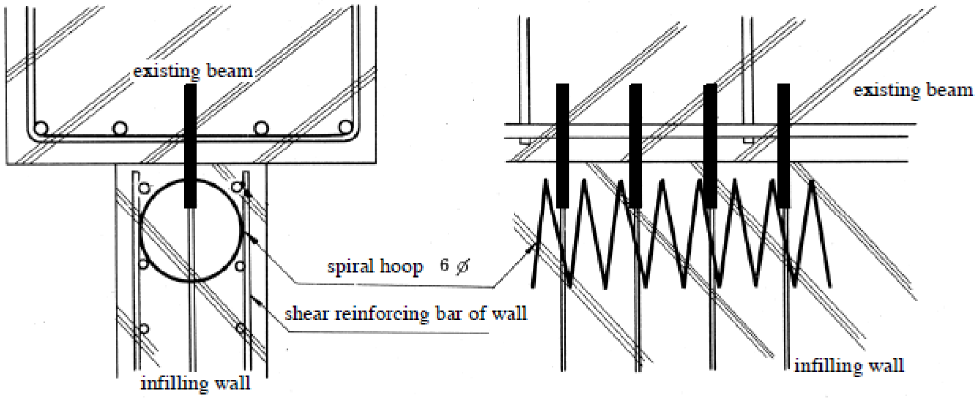
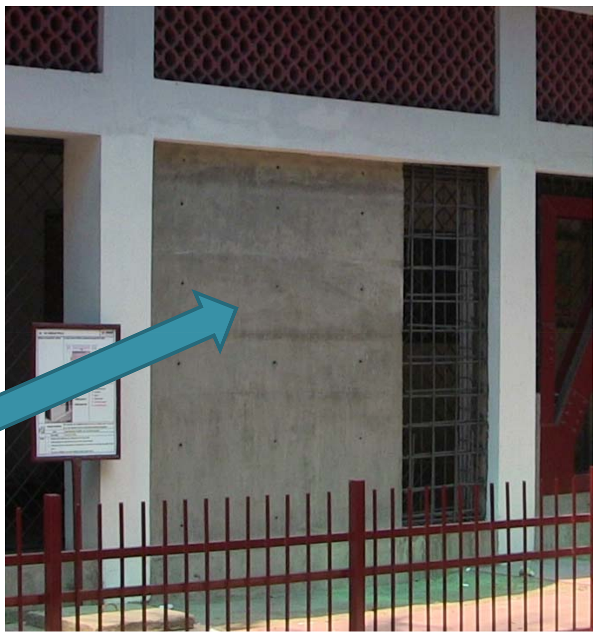


Figure TN.11 Strengthening against splitting with spiral reinforcing bars
(quoted from the figure on page 98 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

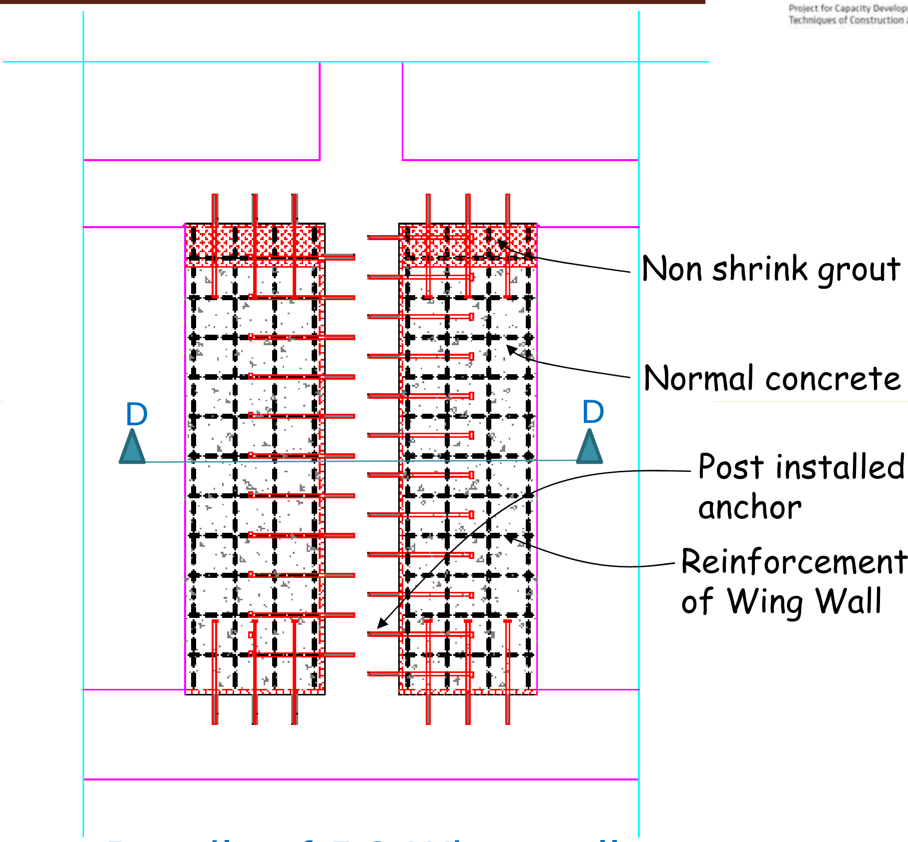
Retrofitting with Shear Wall

Providing RC Shear Wall in a open frame



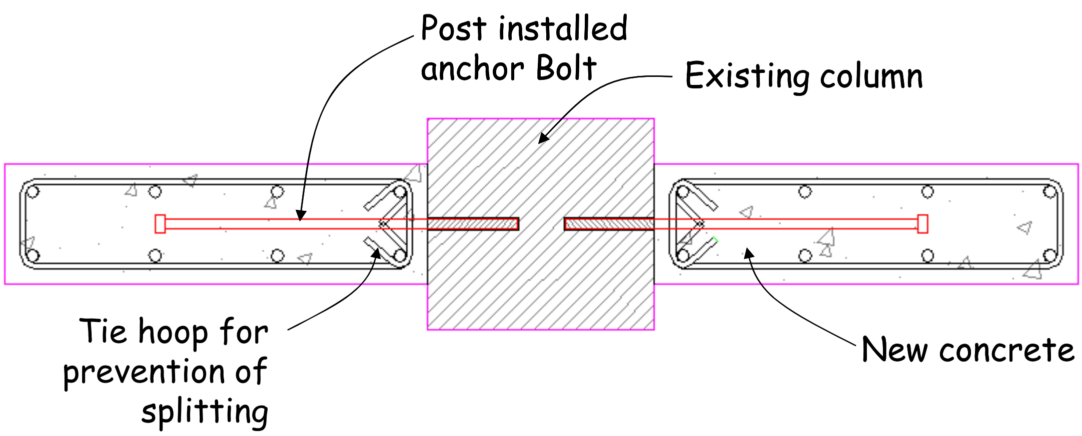
Test Work of CNCRP in 2012

Retrofitting with Wing Wall



Details of RC Wing wall

Retrofitting with Wing Wall



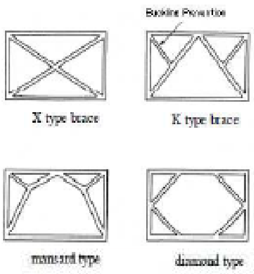
Section D-D

Retrofitting with Wing Wall

Test Work of CNCRP in 2012

Retrofitting with Steel Frame

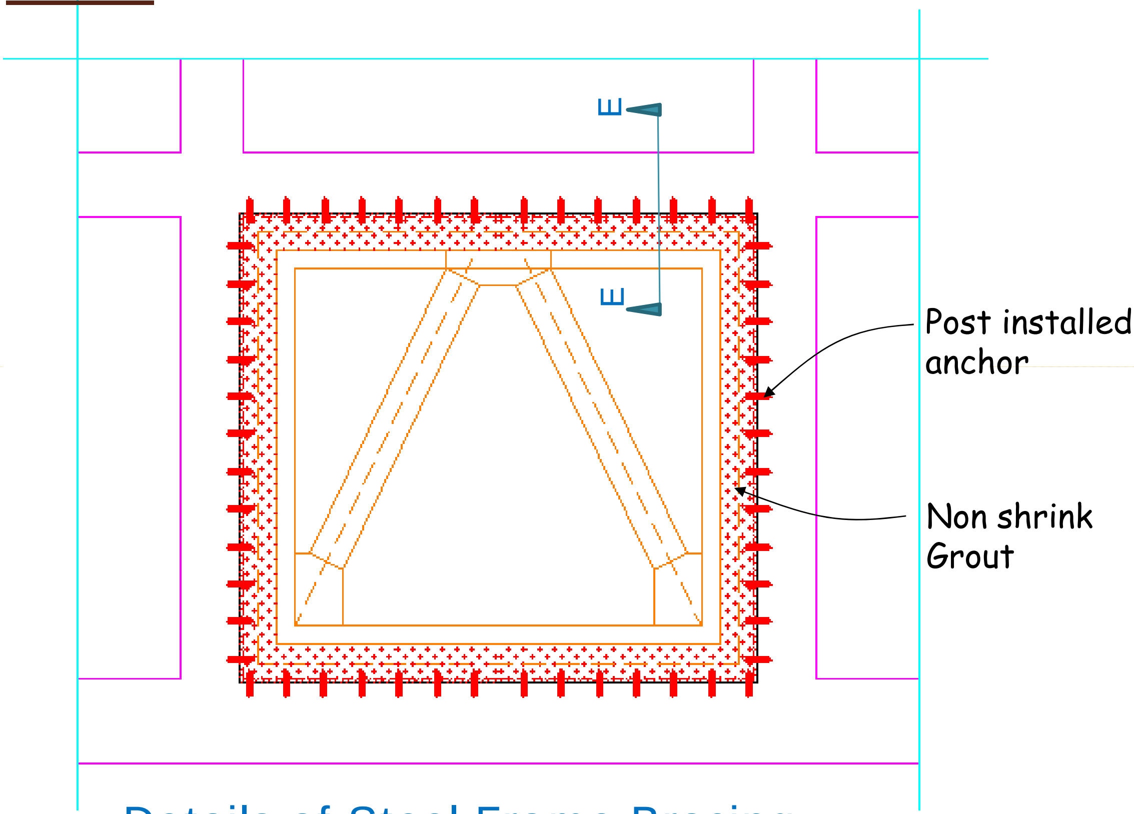
1. Steel Framed Bracing



Ookashiwa Elementary School

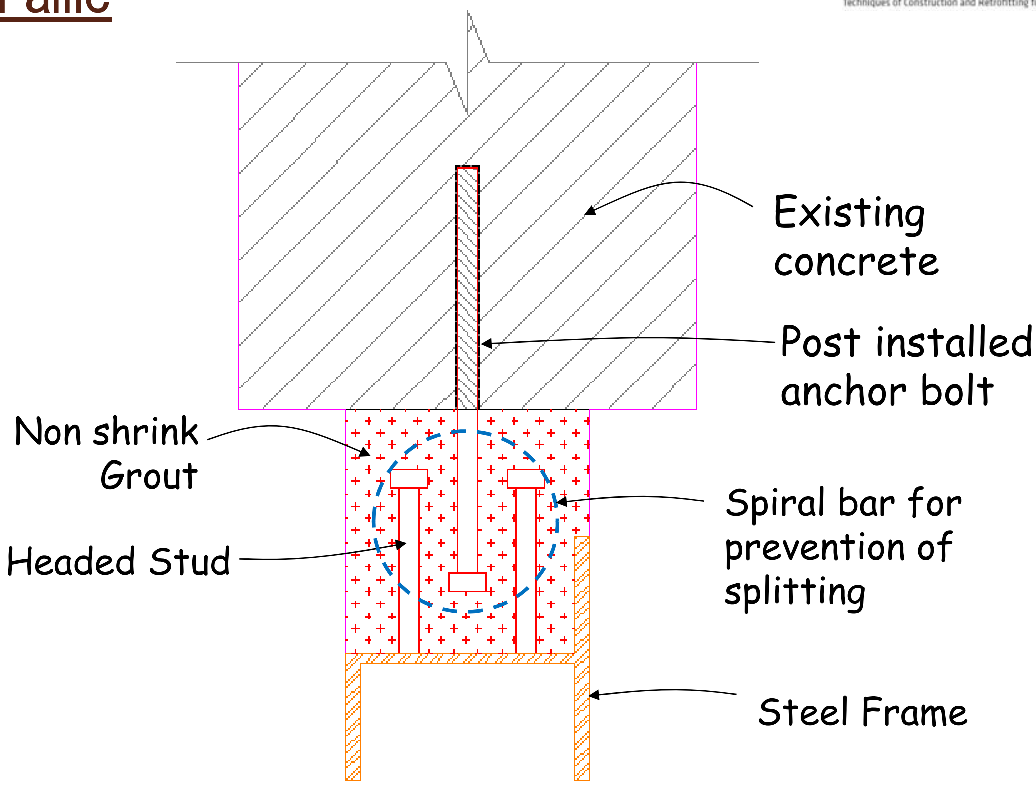
SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Retrofitting with Steel Frame



Details of Steel Frame Bracing

Retrofitting with Steel Frame



Section E-E

Retrofitting with Steel Frame

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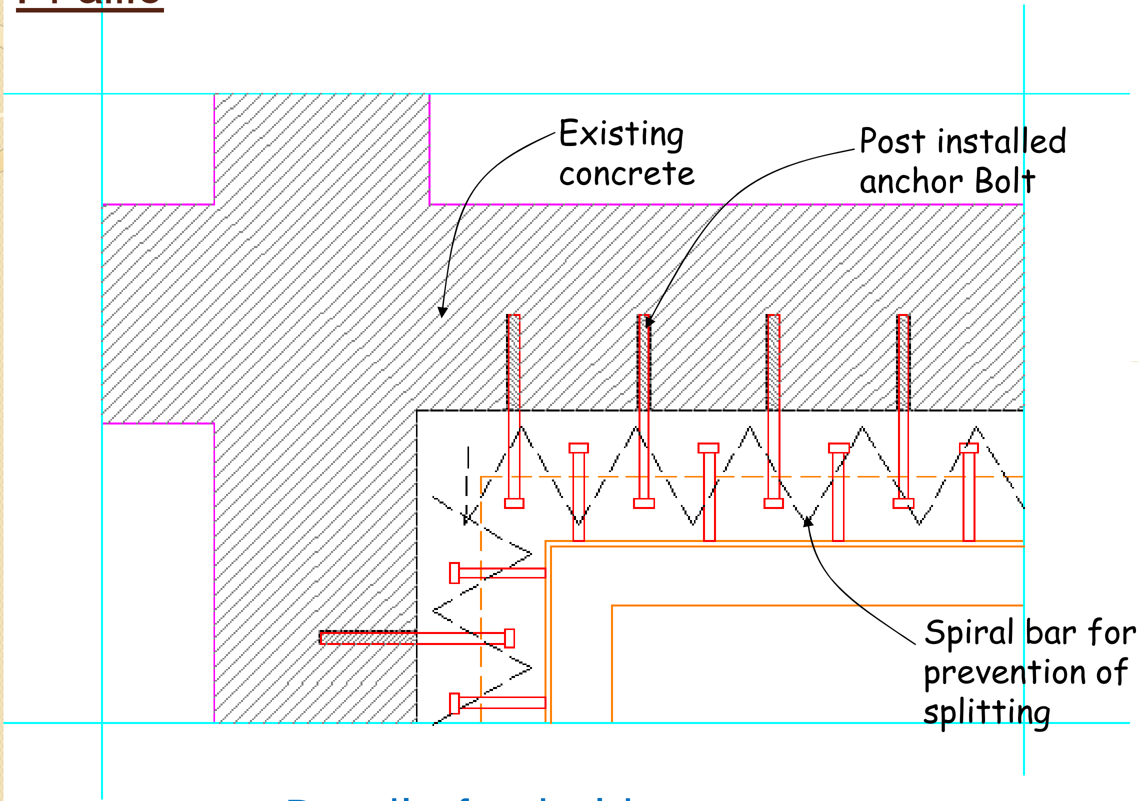


Detail of spiral bar

Retrofitting with Steel Frame

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

Internal Steel Frame Bracing

Test Work of CNCRP in 2012

External Steel Frame Bracing

Test Work of CNCRP in 2013

Connection Details, Test Work 2012

Steel framed bracing

a)Junior High School Building (School Colored)

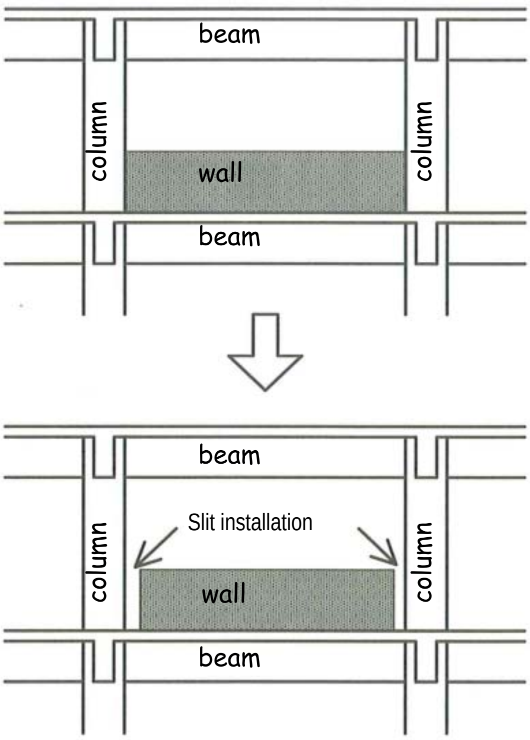
b) Junior High School Building (Designed by Students)

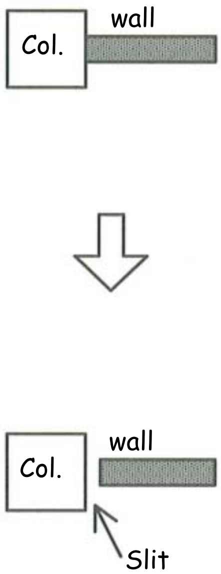
c)University Building (with Exterior Panels)

d)Elementary School Building (with Balconies)

SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

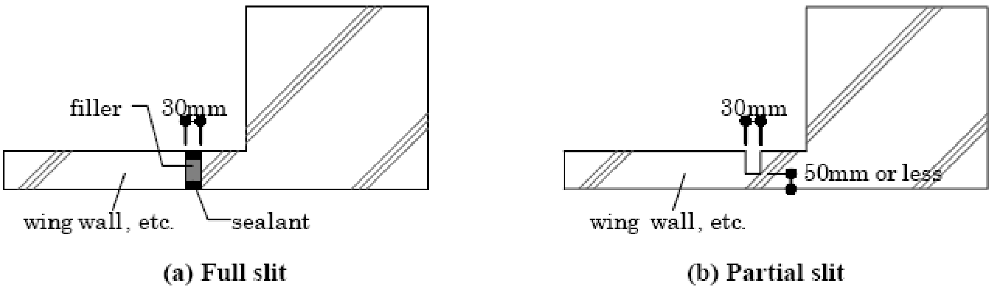
Retrofitting with Structural Slit





SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Retrofitting with Structural Slit



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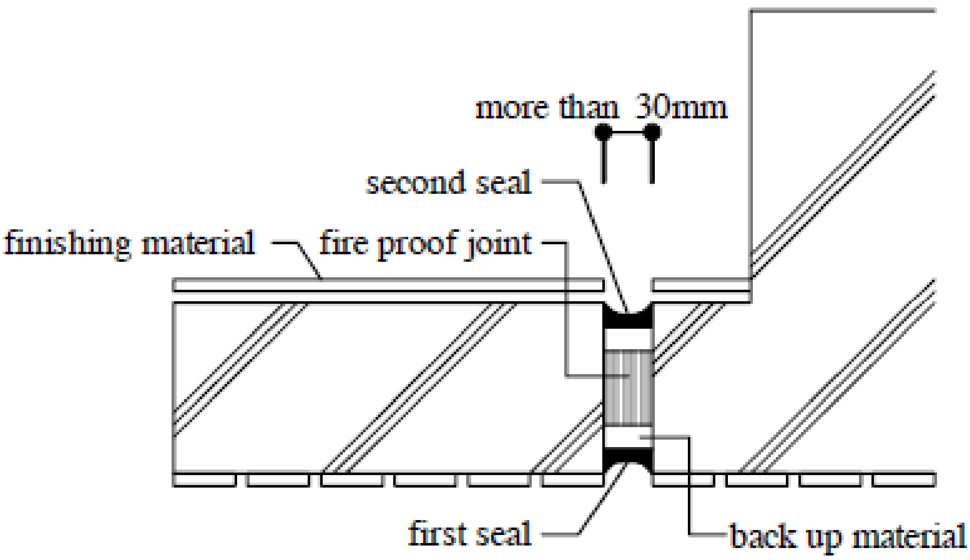
Retrofitting with Structural Slit



Detail of Seismic Slit

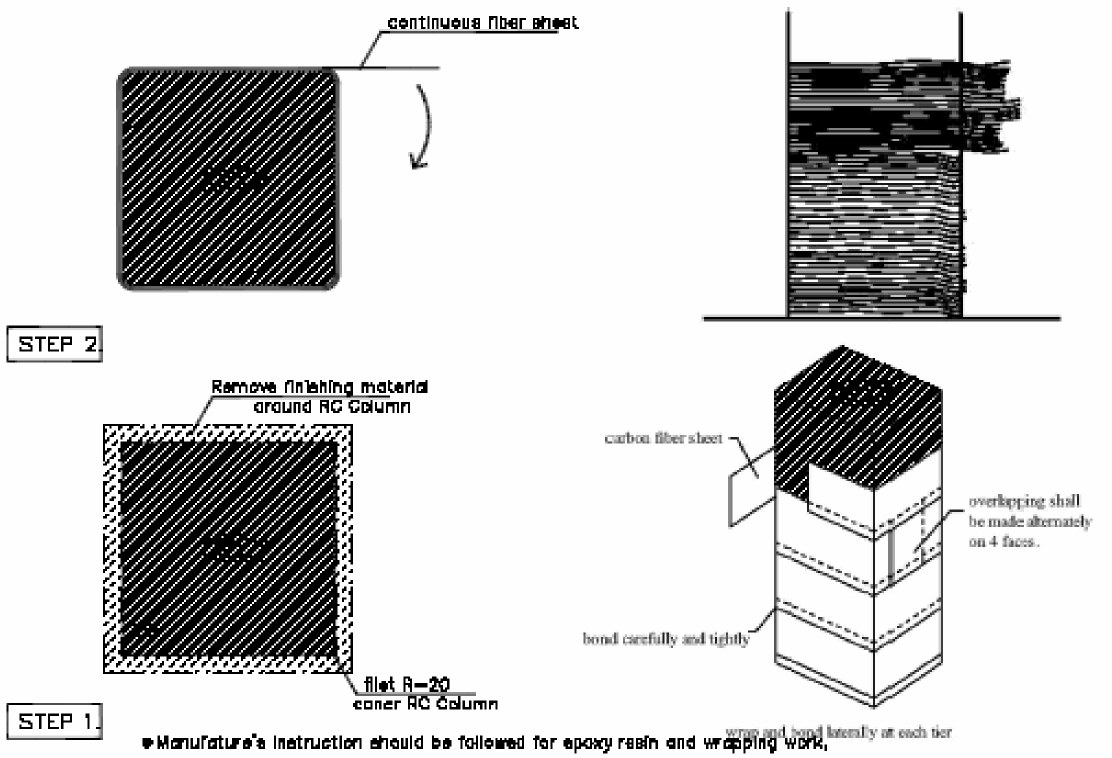
SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Retrofitting with Structural Slit

Seismic Slit is provided at a brick wall

Test Work of CNCRP in 2012

Carbon fiber sheet wrapping



Carbon fiber sheet wrapping

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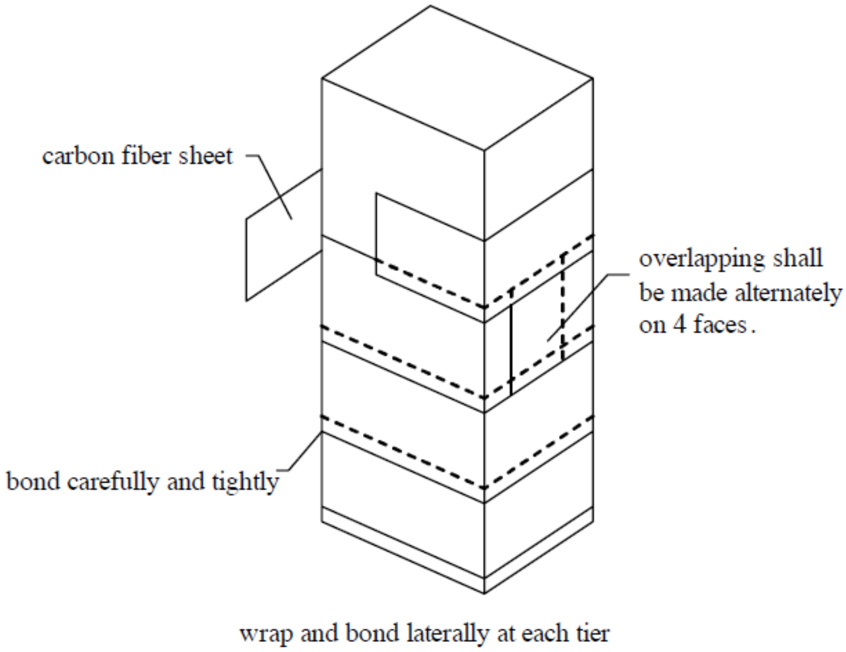

Figure TN.28 Strengthening with carbon fiber sheet wrapping

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Carbon fiber sheet wrapping

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Test Work of CNCRP in 2012

Test Work of CNCRP in 2012

Carbon fiber sheet wrapping

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

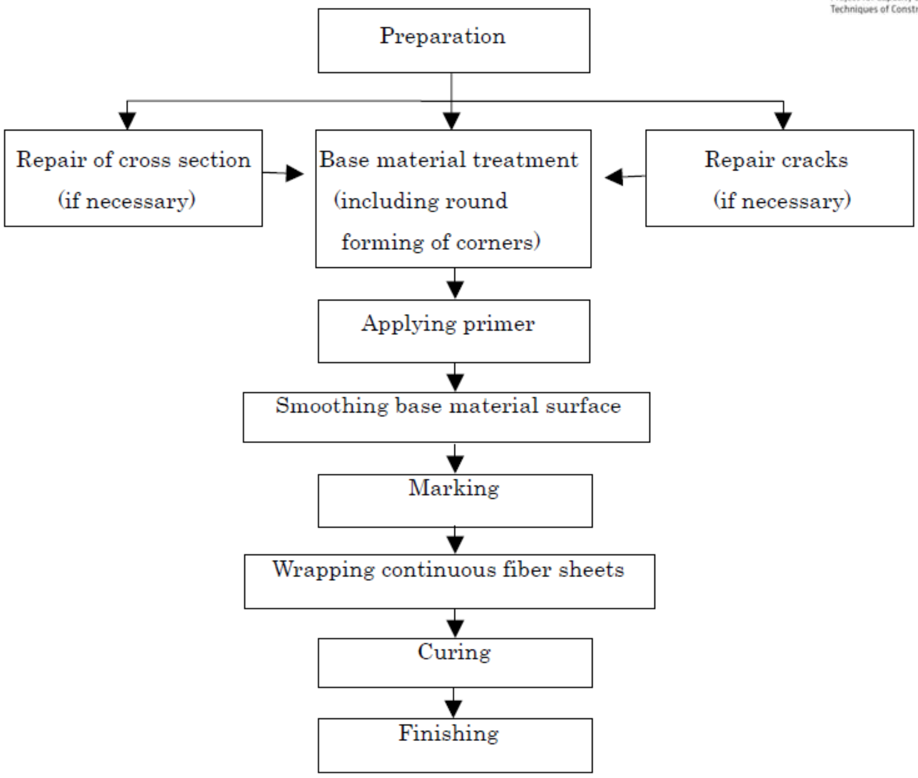
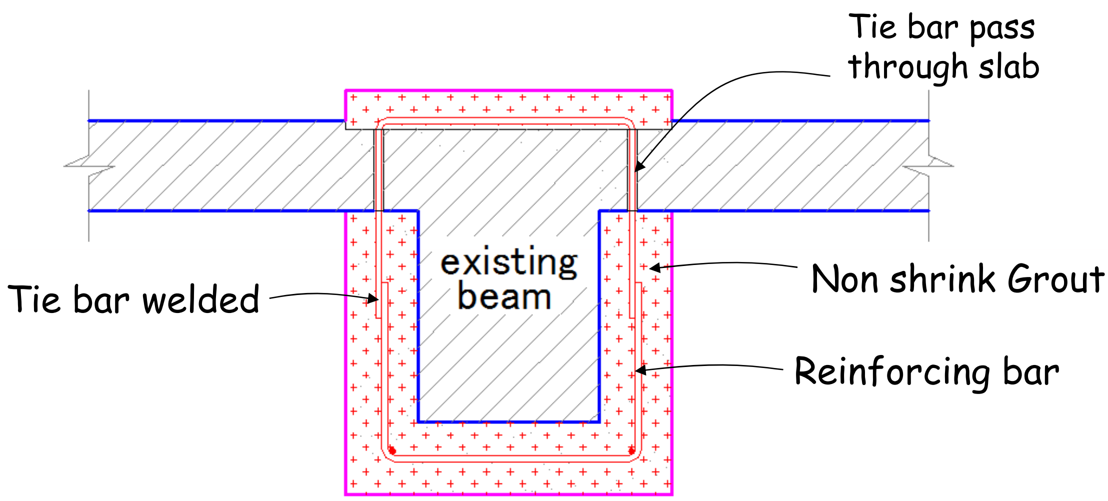


Figure 4.9-1 Flow of standard construction procedure

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Retrofitting with Beam Jacketing

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

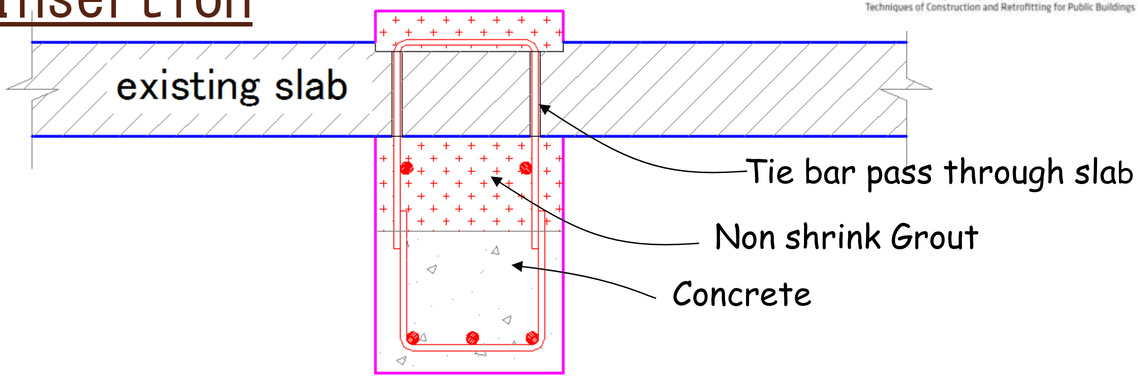


Typical Detail of Beam Jacketing

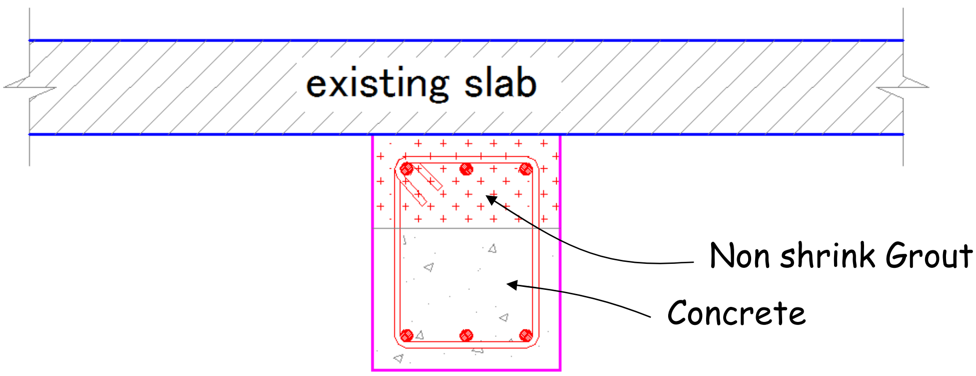
Retrofitting with Beam Jacketing

Test Work of CNCRP in 2013

Retrofitting with Beam Insertion



Typical Detail of Beam Insertion (Option-1)



Typical Detail of Beam Insertion (Option-2)

Retrofitting with Beam Insertion

CNCRP
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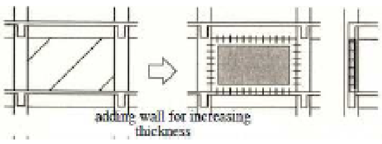
Beam is inserted below existing slab

Test Work of CNCRP in 2013

Methods of Retrofitting

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

3. Increasing Thickness of Existing Shear Wall



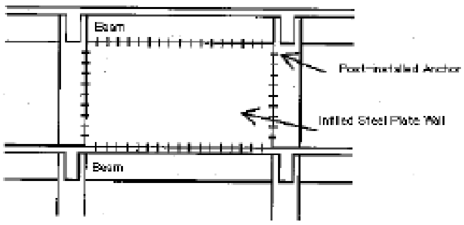
Da Vinch Ginza Building

SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Methods of Retrofitting

Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

4. Infilling Steel Plate Wall into Open Frame



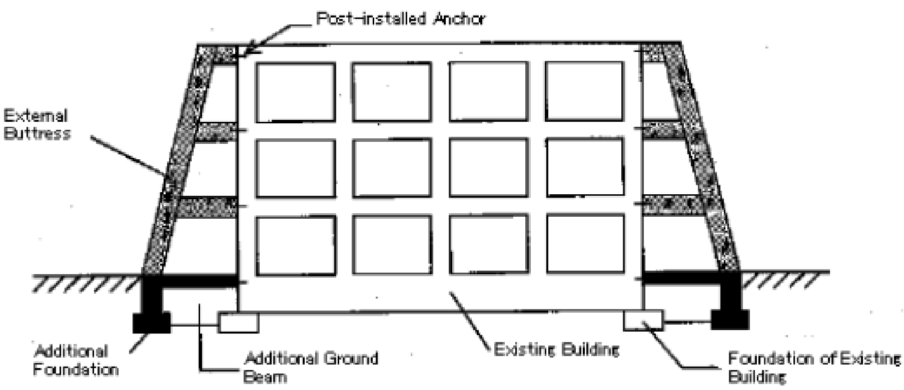
Under Construction

SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Methods of Retrofitting

Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

7. Constructing External Buttress



SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Base Isolation

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

Lead Rubber Bearing with Isolator used

Damper used

Post-Installed Anchor Work

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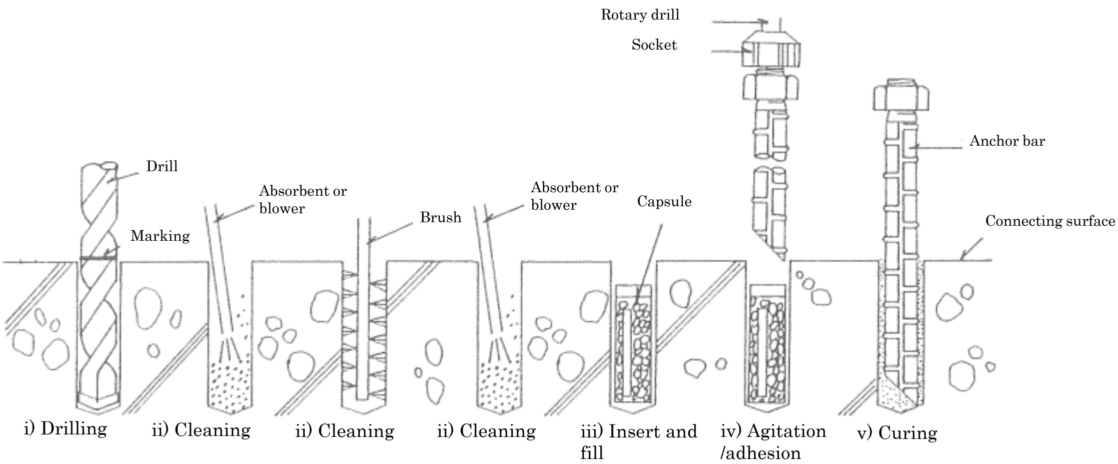

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Post-Installed Anchor Work

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Pressurized Grouting Work

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Thank you very much

SHORT TRAINING COURSE ON SEISMIC ASSESSMENT, RETROFIT DESIGN AND CONSTRUCTION OF RC BUILDING

TITLE OF LECTURE

RETROFITTING DESIGN EXAMPLE OF A REAL STRUCTURE

PRESENTED BY

ANUP KUMAR HALDER

SUB DIVISIONAL ENGINEER

PWDDESIGN DIVISION-V.

&

TEAM MEMBER WORKING TEAM-II

OUTLINE

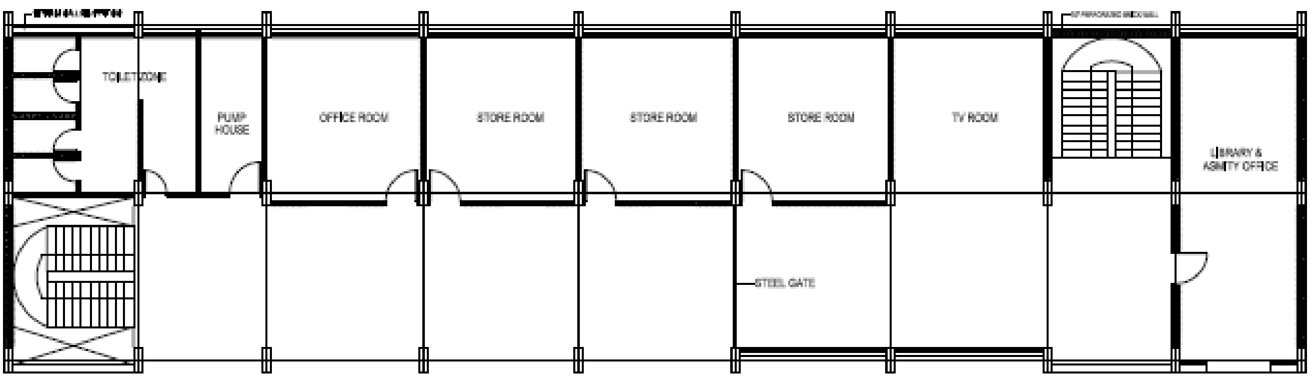
1. BUILDING VIEW/ PLAN/ LAYOUT/ELEVATION
2. INSPECTION FOR BUILDING DATA
3. ASSESSMENT IN X DIRECTION (DETAILS OF STOREY-1)
4. ASSESSMENT IN Y DIRECTION (DETAILS OF STOREY-1)
5. C, F VALUE IN X DIRECTION FLOOR WISE
6. CALCULATION OF DEMAND
7. COLUMN JACKETING
8. WING WALL
9. SHEAR WALL
10. CHECK FOR PERFORMANCE OF SW
11. CARBON FIBRE WRAPING
12. STEEL BRACING
13. SELECTION OF METHOD

BUILDING VIEW

ANNEX BUILDING OF MULTISTORIED GARAGE CUM OFFICE BHABAN.

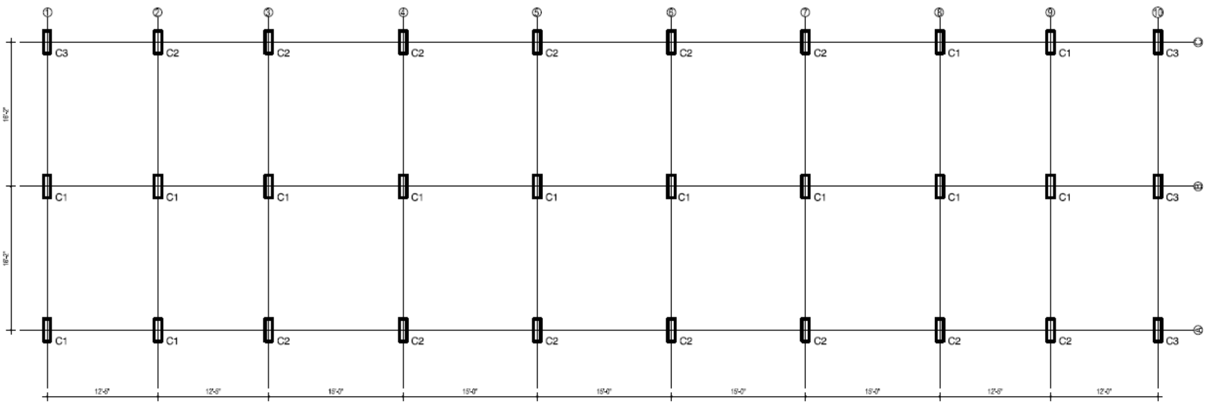
BUILDING PLAN

ANNEX BUILDING OF MULTISTORIED GARAGE CUM OFFICE BHABAN

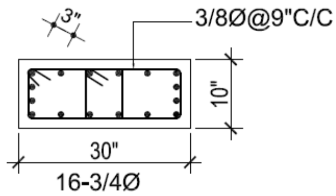


BUILDING LAYOUT

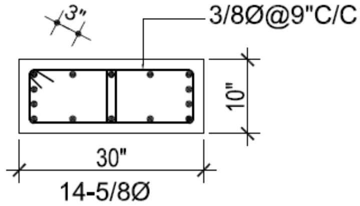
ANNEX BUILDING OF MULTISTORIED GARAGE CUM OFFICE BHABAN



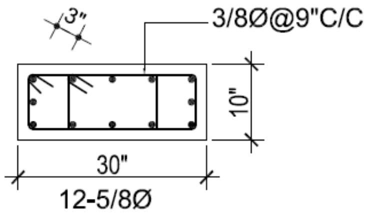
COLUMN LAY OUT PLAN



C-1



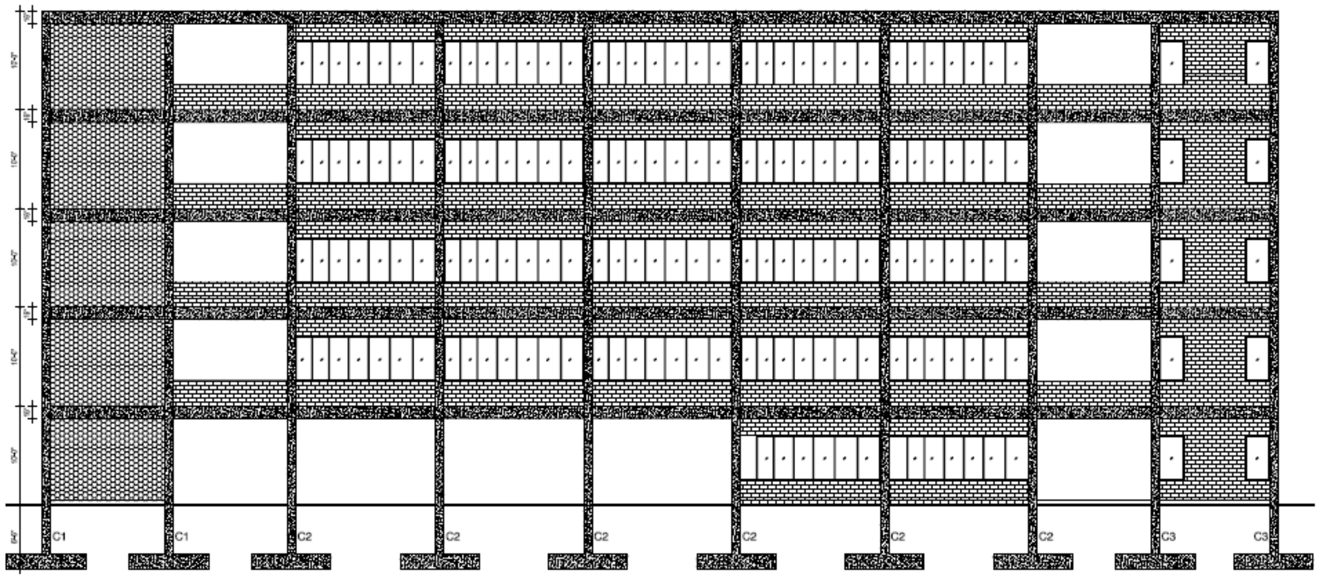
C-2



C-3

ELEVATION GRID A-A

ANNEX BUILDING OF MULTISTORIED GARAGE CUM OFFICE BHABAN



ELEVATION GRID A-A

INSPECTION

ANNEX BUILDING OF MULTISTORIED GARAGE CUM OFFICE BHABAN

BUILDING DATA

NAME	ANNEX BUILDING OF MULTISTORIED GARAGE CUM OFFICE BHABAN.
BUILDING USE	OFFICE
STRUCTURE TYPE	R.C.C FRAMED STRUCTURE
YEAR OF CONSTRUCTION	1985
CONCRETE f’c	9.2 Mpa (DESIGN f’c=13.7Mpa)
REBAR fy	275 Mpa
TOTAL STOREY	5(FIVE)
FLOOR AREA	377.38 Sqm
FOUNDATION TYPE	SHALLOW /DEPTH 4’-6” FROM EGL
BEARING CAP	1.00 TSF

BUILDING DATA cont…..

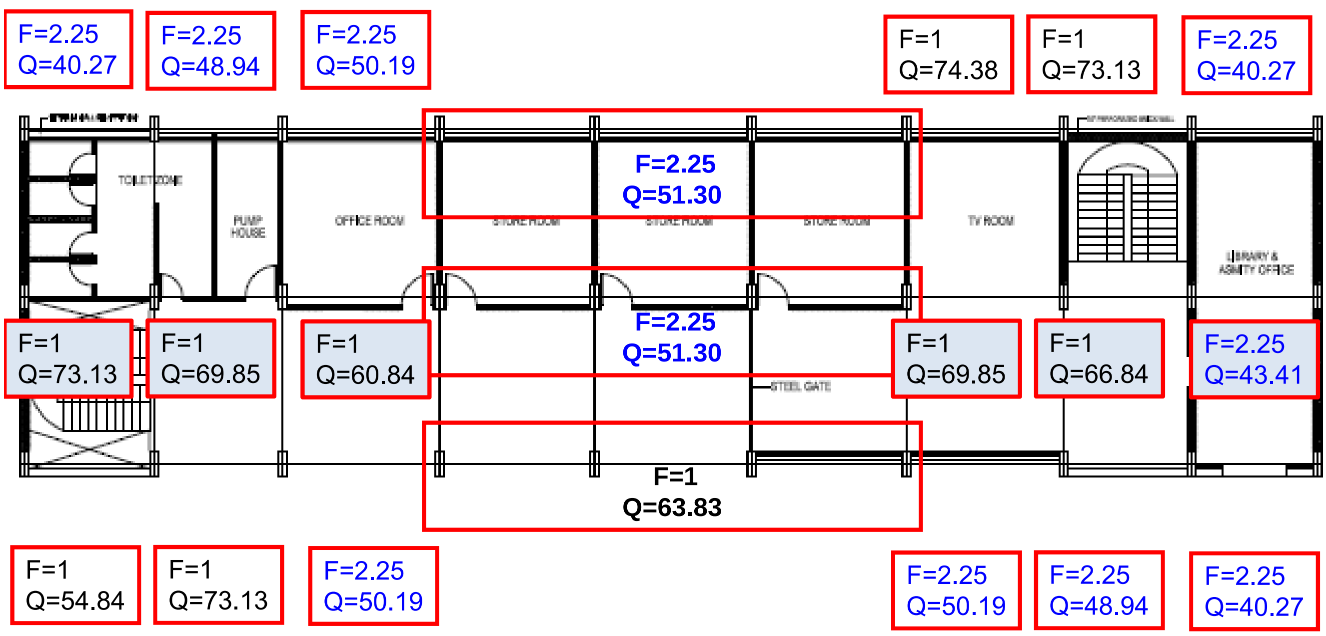
Material Properties:	
f'c(N/mm2)	9.20
σy(N/mm2)	275.00
σwy(N/mm2)	275.00

Unit Area weight(Roof):			
1. Live Load		0.30	kN/Sqm
2. Brick Wall		0.00	kN/Sqm
3. Slab weight & Floor Finish		3.85	kN/Sqm
5. SW(Column+Beam)		2.15	kN/Sqm
	w =	6.3	kN/Sqm

Unit Area weight(Typical Floor):			
1. Live Load		0.80	kN/Sqm
2. Brick Wall		4.00	kN/Sqm
3. Slab weight & Floor Finish		3.85	kN/Sqm
5. SW(Column+Beam)		2.15	kN/Sqm
	w =	10.8	kN/Sqm

FLOOR AREA	377.38	Sqm

BUILDING ASSESSMENT (X DIRECTION STORY 1)



F=2.25

QT=(51.30*8)+(40.27*3)+(48.94*2)+(50.19)*3+43.41=822.89 KN

C=822.89/18692.2=0.04

E_O CALCULATION (X DIR STORY 1)

STRENGTH DOMINANT STRUCTURE

Σwi= 18692.2 kN

Direction	Story	GN	Q	C	ΣQ	C1	F	E0-1	E0-2	Ctu	
X	1	1	0.0	0.00	0.00	0.000	0.80				
		2	874.3	0.05	1466.96	0.078	1.00		0.078	0.08	
		3	0.0	0.00	0.00	0.000	1.10				
		4	0.0	0.00	0.00	0.000	1.20				
		5	0.0	0.00	0.00	0.000	1.27				
		6	0.0	0.00	0.00	0.000	1.40				
		7	0.0	0.00	0.00	0.000	1.50				
		8	0.0	0.00	0.00	0.000	1.75				
		9	0.0	0.00	0.00	0.000	2.00				
		10	823.1	0.04	823.10	0.044	2.25		0.099	0.04	
		11	0.0	0.00	0.00	0.000	2.60				
		12	0.0	0.00	0.00	0.000	3.00				
		13	0.0	0.00	0.00	0.000	3.20				
		ΣQ	1697.4		MAX_E0	0.044	2.25		0.099		

DUCTILITY DOMINANT STRUCTURE

$$E_O = \sqrt{(1*0.05)^2+(2.25*0.04)^2} = 0.11$$

ASSESSMENT SUMMARY (X DIRECTION)

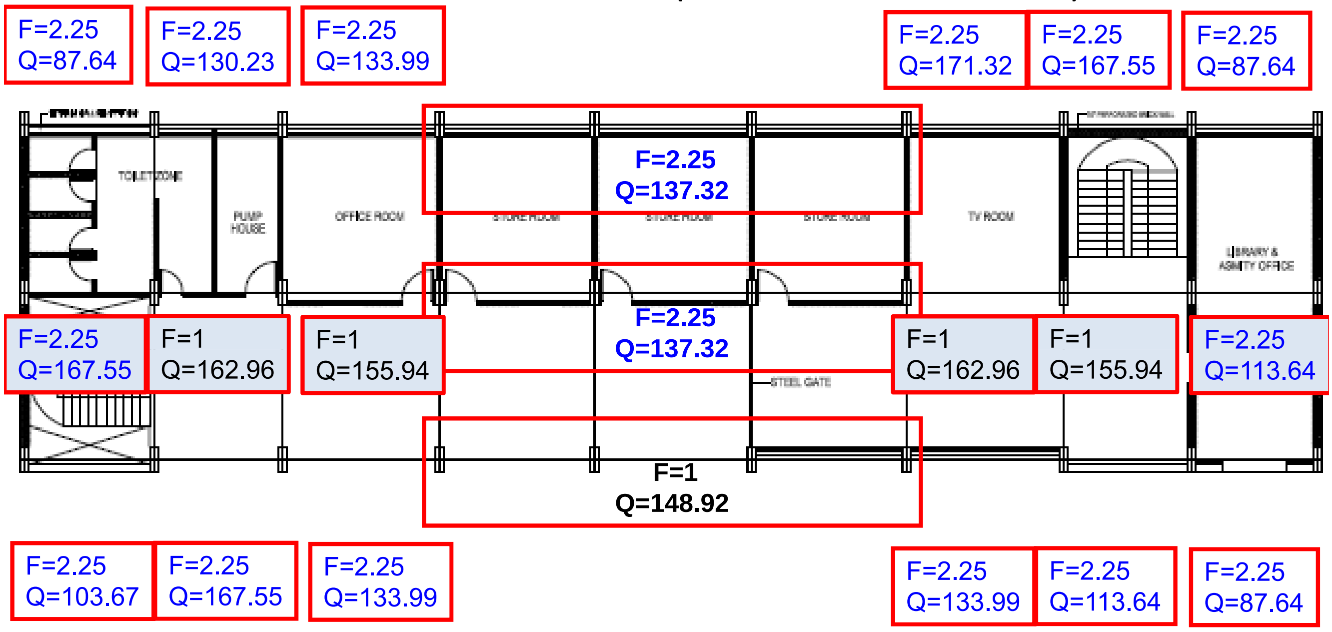
Direction	Story	C	F	Failure Mode	Eo	T	S_D	Is	$C_{TU}\cdot S_D$	Result	Adoption	Eq
X	5											5
		0.339	2.25		0.458			0.458	0.203	OK		5
		0.478	1.00		0.540	1.000	1.000	0.540	0.203	OK		4
		0.339	2.25									
		0.722	1.00		[0.433]			0.433	[0.433]	OK		5
	4											5
		0.147	2.25		[illegible]			[illegible]	0.098	NG		5
		0.200	1.00		0.258	1.000	1.000	[illegible]	[illegible]	[illegible]		4
		0.147	2.25									
		0.307	1.00		[0.155]			0.155	[0.155]	NG		5
	3											5
		0.101	2.25		0.170			0.170	0.076	NG		5
		0.133	1.00		0.197	1.000	1.000	0.197	0.076	NG		4
		0.101	2.25									
		0.206	1.00		[0.154]			0.154	[0.154]	NG		5
	2											5
		0.080	2.25		0.155			0.155	0.069	NG		5
		0.095	1.00		[illegible]	[illegible]	[illegible]	0.110	0.069	NG		4
		0.080	2.25									
		0.153	1.00		[0.131]			0.131	[0.131]	NG		5
	1											5
		0.044	2.25		0.099			0.099	0.044	NG		5
		0.047	1.00		0.110	1.000	1.000	0.110	0.044	NG		4
		0.044	2.25									
		0.078	1.00		[0.078]			0.078	[0.078]	NG		5

DUCTILITY DOMINANT

STRENGTH DOMINANT

Is CONSIDERED FOR RETROFITTING

BUILDING ASSESSMENT (Y DIRECTION STORY 1)



F=2.25

QT=(148.92*4)+(155.94*2)+(162.96*2)=1233.48 KN

C=1233.48/18692.2=0.06

CALCULATION OF Eo (STRENGTH DOMINENET STRUCTURE)

STRENGTH DOMINANT STRUCTURE

N+1/N+i= 1.000

Σwi= 18692.2 kN

Direction	Story	GN	Q	C	ΣQ	C1	F	E0-1	E0-2	Ctu	
Y	1	1	0.0	0.00	0.00	0.000	0.80				
		2	1233.5	0.07	3320.48	0.178	1.00		0.178	0.18	
		3	0.0	0.00	0.00	0.000	1.10				
		4	0.0	0.00	0.00	0.000	1.20				
		5	0.0	0.00	0.00	0.000	1.27				
		6	0.0	0.00	0.00	0.000	1.40				
		7	0.0	0.00	0.00	0.000	1.50				
		8	0.0	0.00	0.00	0.000	1.75				
		9	0.0	0.00	0.00	0.000	2.00				
		10	2898.6	0.16	2898.61	0.155	2.25		0.349	0.16	
		11	0.0	0.00	0.00	0.000	2.60				
		12	0.0	0.00	0.00	0.000	3.00				
		13	0.0	0.00	0.00	0.000	3.20				
		ΣQ	4132.1	MAX_E0		0.155	2.25		0.349		

DUCTILITY DOMINANT STRUCTURE

$$E_O = \sqrt{(1*0.07)^2+(2.25*0.16)^2}=0.36$$

ASSESSMENT SUMMARY (Y DIRECTRION)

Seismic demand index							Iso=	0.30	$C_{TU}\cdot S_D$=	0.15		
Direction	Story	C	F	Failure Mode	Eo	T	S_D	Is	$C_{TU}\cdot S_D$	Result	Adoption	Eq
Y	5					1.000	1.000					5
		1.789	1.50		1.610			1.610	1.073	OK		5
		0.646	1.50		1.649			1.649	0.686	OK		4
		1.143	2.25									
												5
	4					1.000	1.000					5
		0.746	1.20		0.596			0.596	0.497	OK		5
		0.391	1.20		0.651			0.651	0.254	OK		4
		0.381	2.25									
												5
	3					1.000	1.000					5
		0.515	1.20		0.463			0.463	0.386	OK		5
		0.268	1.20		0.507			0.507	0.198	OK		4
		0.265	2.25									
												5
	2					1.000	1.000					5
		0.214	2.00		0.367			0.367	0.184	OK		5
		0.189	1.00		0.355			0.355	0.184	OK		4
		0.214	2.00									
		0.343	1.00		[0.294]			0.294	[0.294]	NG		5
	1					1.000	1.000					5
		0.155	2.25		0.349			0.349	0.155	OK		5
		0.066	1.00		0.355			0.355	0.155	OK		4
		0.155	2.25									
		0.178	1.00		[0.178]			0.178	[0.178]	NG		5

ASSESSMENT IN Y DIR STORY 1

Seismic demand index								Iso=	0.30	$C_{TU}\cdot S_D$=	0.15	
Y	1					1.000	1.000					5
		0.155	2.25		0.349			0.349	0.155	OK		5
		0.066	1.00		0.355			0.355	0.155	OK		4
		0.155	2.25									
		0.178	1.00		[0.178]			0.178	[0.178]	NG		5

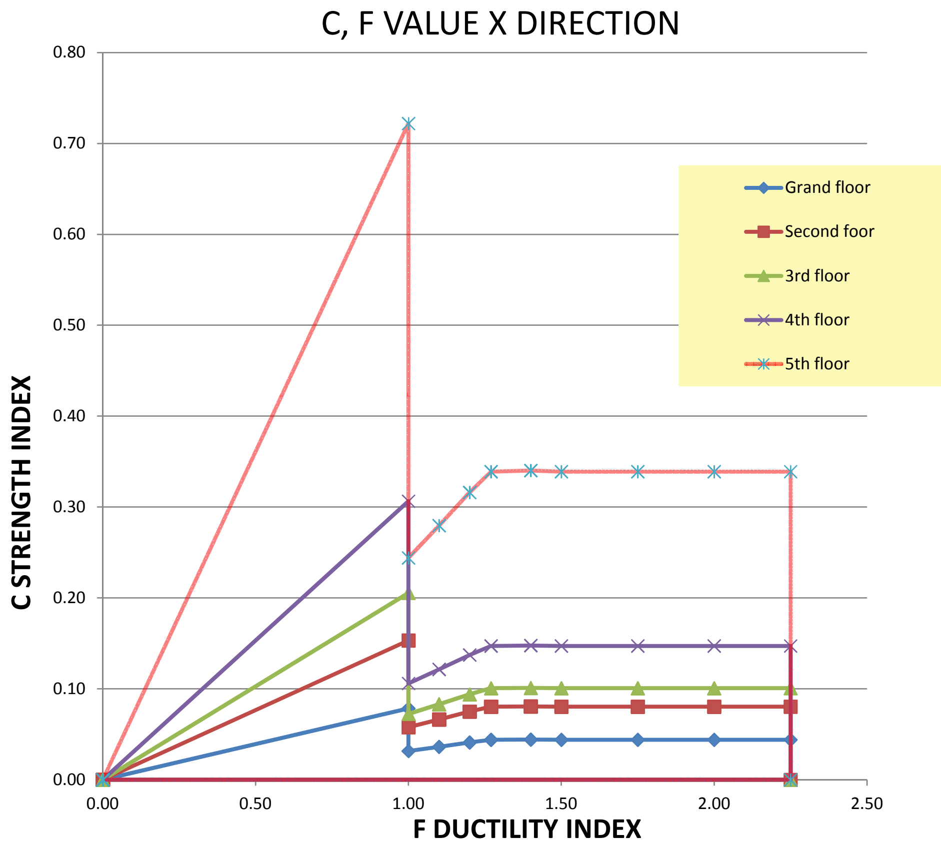
N+1/N+i= 1.000

Σwi= 18692.2 kN

Direction	Story	GN	Q	C	ΣQ	C1	F	E0-1	E0-2	Ctu
Y	1	1	0.0	0.00	0.00	0.000	0.80			
		2	1233.5	0.07	3320.48	0.178	1.00		0.178	0.18
		3	0.0	0.00	0.00	0.000	1.10			
		4	0.0	0.00	0.00	0.000	1.20			
		5	0.0	0.00						
		6	0.0	0.00						
		7	0.0	0.00						
		8	0.0	0.00						
		9	0.0	0.00	0.00	0.000	2.00			
		10	2898.6	0.16	2898.61	0.155	2.25		0.349	0.16
		11	0.0	0.00	0.00	0.000	2.60			
		12	0.0	0.00	0.00	0.000	3.00			
		13	0.0	0.00	0.00	0.000	3.20			
		ΣQ	4132.1		MAX_E0	0.155	2.25		0.349	

DUCTILITY DOMINANT STRUCTURE

$E_O = \sqrt{(1*0.07)^2+(2.25*0.16)^2} = 0.36$



CALCULATION OF DEMAND

- $I_{SO}=E_O x S_D x T=C1xFXS_D xT=C1=\sum Q1/W$
- $I_{SX}=E_O x S_D x T=C2xFXS_D xT=C2=\sum Q2/W$
- CONSIDERING NO CHANGE IN THE SYSTEM WITH ($FXS_D xT$)
- $I_{SO}-I_{SX}=\sum Q1/W-\sum Q2/W$
- $(I_{SO}-I_{SX})XW=\sum Q1-\sum Q2$=REQUIRED SHEAR CAPACITY
- $I_{SO}=0.3$
- $I_{SX}=0.078$
- $I_{SY}=0.178$
- W=18692.2 KN
- SHEAR REQUIREMENT IN X=(0.3-0.078)X18692.2=4150KN
- SHEAR REQUIREMENT IN Y=(0.3-0.178)X18692.2=2280KN

COLUMN JACKETING

When $0.4b \cdot D \cdot F_{c1} \geq N \geq 0$,

$$M_u = a_t \cdot \sigma_y \cdot g + a_{t2} \cdot \sigma_{y2} \cdot g_2 + 0.5 \cdot N \cdot D_2 \cdot \left(1 - \frac{N}{b_2 \cdot D_2 \cdot F_{c1}}\right)$$

Old rebar

Concrete

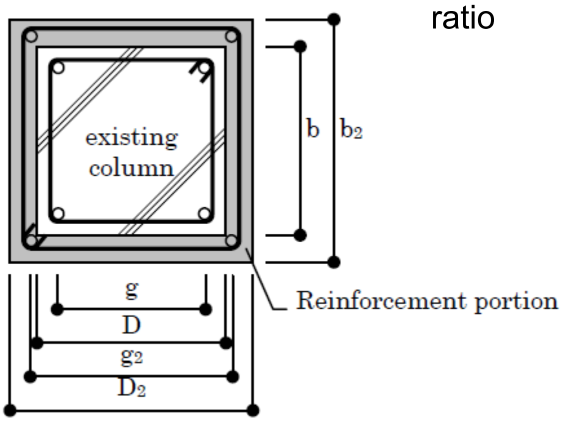
Area of tension rebar in jacketing part

$$Q_{su} = \phi \left\{ \frac{0.053 \cdot p_{t2}^{0.23} \cdot (F_{c1} + 18)}{M / (Q \cdot d_2) + 0.12} + 0.85 \sqrt{p_w \cdot \sigma_{wy} + p_{w2} \cdot \sigma_{wy2}} + 0.1 \frac{N}{b_2 \cdot D_2} \right\} \times 0.8 \cdot b_2 \cdot D_2$$

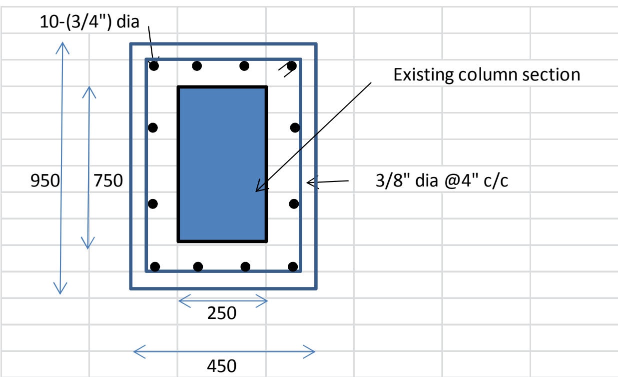
Shear reinforcement

Axial force ratio

Concrete strength, main rebar ratio, slenderness



COLUMN JACKETING cont...



- SHEAR REQUIREMENT IN X=4150KN

X-DIRECTION
Q =150KN
Qpre=70KN
Qgain=80KN
NOT GOOD

- SHEAR REQUIREMENT IN Y=2280KN

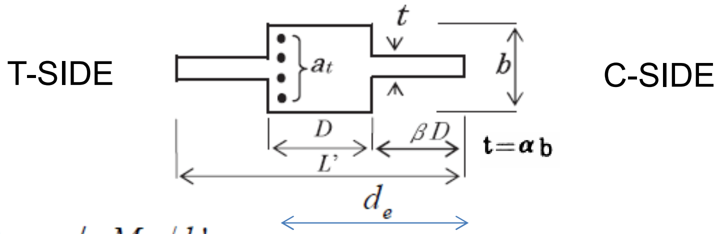
Y-DIRECTION
Q =242KN
Qpre=130KN
Qgain=112KN
Appx. 20Column

WING WALL

- CONTRIBUTION OF TENSION SIDE WING WALL IGNORED

$$M_u = (0.9+\beta)\cdot a_t \cdot \sigma_y \cdot D + 0.5N\cdot D\left\{1+2\beta - \frac{N}{\alpha_e \cdot b \cdot D \cdot F_{c1}}\left(\frac{a_t \cdot \sigma_y}{N}+1\right)^2\right\}$$

$$\alpha_e = (1+2\alpha\cdot\beta)/(1+2\beta)$$



$$Q_{mu} = \phi \cdot M_u / h'$$

$$Q_{su} = \phi\left\{\frac{0.053\cdot p_{te}^{0.23}\cdot(F_c+18)}{M/(Q\cdot d_e)+0.12} + 0.85\sqrt{p_{we}\cdot\sigma_{wy}} + 0.1\sigma_{oe}\right\}\cdot b_e \cdot j_e$$

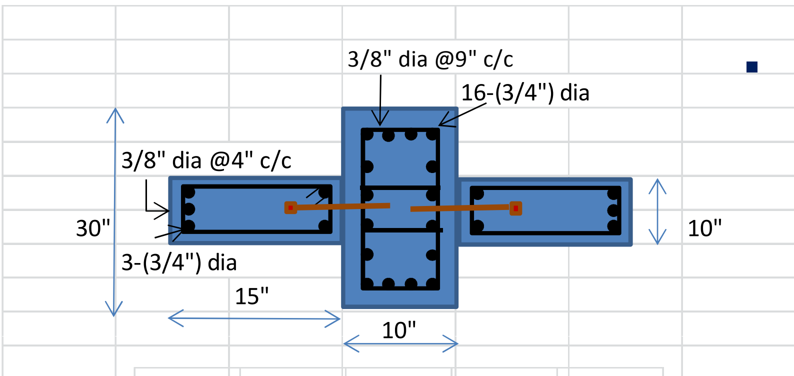
$$p_{te} = 100a_t/(b_e \cdot d_e) \qquad p_{we}\cdot\sigma_{wy} = p_w\cdot\sigma_{wy}(b/b_e) + p_{sh}\cdot\sigma_{sy}(t/b_e)$$

LATERAL REBAR RATIO(EXISTING COL+ WING WALL)

$$\sigma_{oe} = N/(b_e \cdot j_e)$$

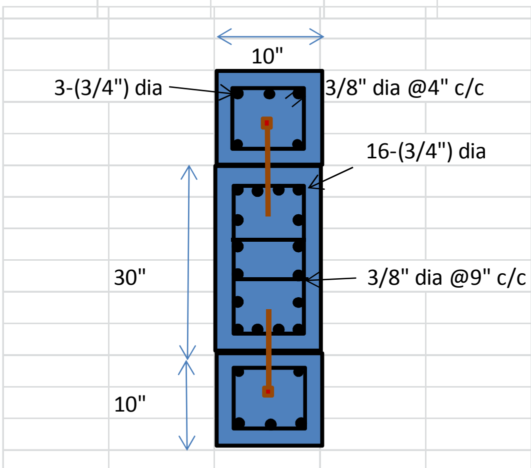
$$b_e = \alpha_e \cdot b$$

WING WALL cont…



- SHEAR REQUIREMENT IN X=4150KN

X-DIRECTION
Q =300KN
Qpre=70KN
Qgain=230KN
Appx. 18Column



- SHEAR REQUIREMENT IN Y=2280KN

Y-DIRECTION
Q =270KN
Qpre=130KN
Qgain=140KN
Appx. 17Column

SHEAR WALL CALCULATION

SHEAR STRENGTH OF SW

$$_{W}Q_{su} = \min\left\{ {}_{W}Q'_{su} + 2\cdot\alpha\cdot Q_c\ ,\ Q_j + {}_{p}Q_c + \alpha\cdot Q_c \right\}$$

Shear force of column

Direct shear strength at top of col

Shear strength of infill panel

Shear connector

$$_{W}Q'_{su} = \max\left(p_w \cdot {}_{W}\sigma_y , F_{cw}/20 + 0.5 p_w \cdot {}_{W}\sigma_y \right)\cdot t_W \cdot l'$$

Wall reinforcement ratio and yield strength

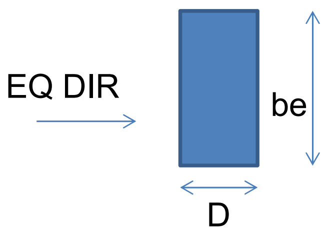
Wall thickness & clear span

SHEAR STRENGTH COLUMN

$$_{p}Q_c = K_{\min}\cdot\tau_o\cdot b_e\cdot D$$

Q_j = Sum of the shear strengths of connectors underneath the beam.

SHEAR WALL CALCULATION cont..

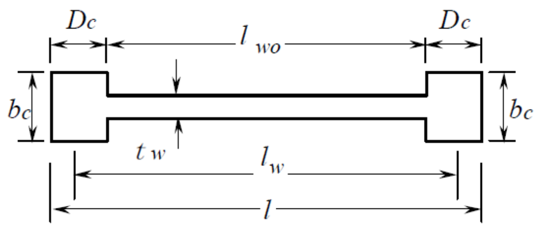


$\tau_o = 0.66F_{c1}$ in case $0.66 < \sigma$

$\sigma_o = N / (b_e \bullet D)$

$\sigma = p_g \cdot \sigma_y + \sigma_o$

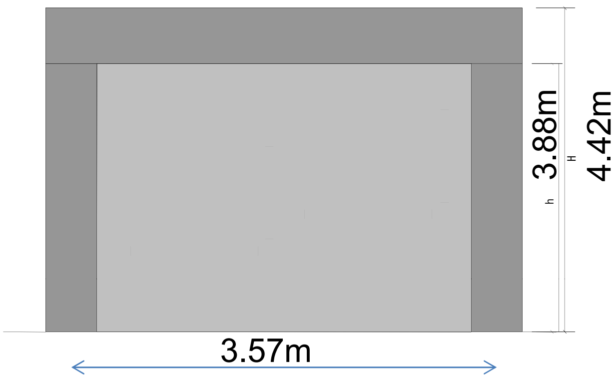
$p_g = a_g / b_e \cdot D$

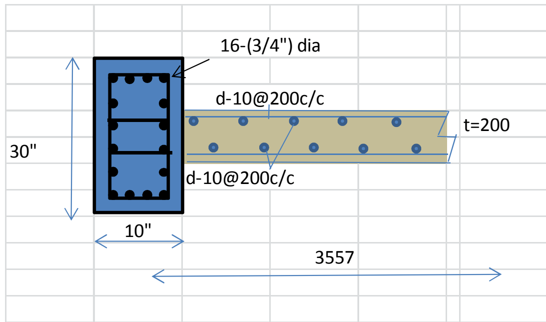


FLEXURAL STRENGTH OF SW

$$_{W}M_u = a_t\cdot\sigma_{sy}\cdot l_W + 0.5\sum(a_{wy}\cdot\sigma_{wy})\cdot l_W + 0.5N\cdot l_W$$

SHEAR WALL CALCULATION cont..





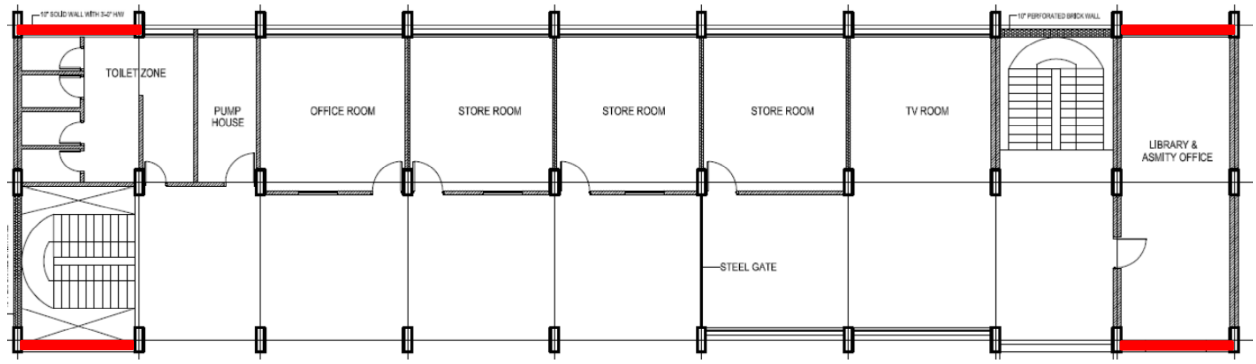
$_{W}Q_{su}$ =1400KN (X-DIRECTION)

$_{W}Q_{su}$ =1500KN (Y-DIRECTION)

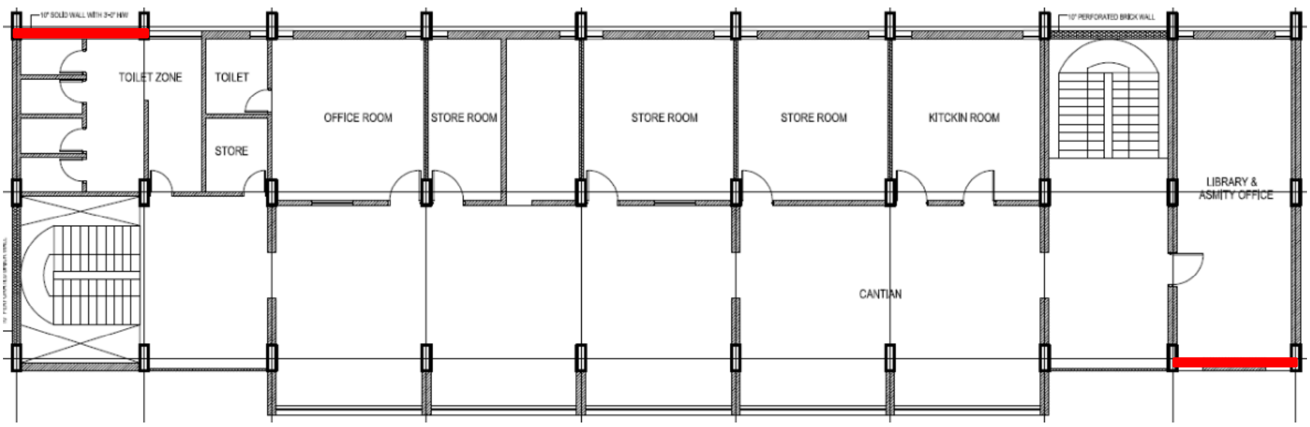
CALCULATION FOR SW(X- DIR)

Direction	Story	C	F	Eo	T	S_D	Is	$C_{TU} \cdot S_D$	Result	Adoption	Eq	WT (KN)	n+1/(n+i)	SW Cap (KN)	Req SW No=(Is0-Is)XWt/SW cap	No of SW
	5										5					
		0.339	2.25	0.458			0.458	0.203	OK		5					
		0.478	1.00						OK		4					
		0.339	2.25	0.540	1.000	1.000	0.540	0.203								
		0.722	1.00	[0.433]			0.433	[0.433]	OK		5	2377.494	0.6	840	-0.4	Not Required
	4										5					
		0.147	2.25	0.221			0.221	0.098	NG		5					
		0.200	1.00													
		0.147	2.25	0.258	1.000	1.000	0.258	0.980	NG		4					
		0.307	1.00	[0.155]			0.155	[0.155]	NG		5	6453.198	0.67	933.3333333	0.6	1
	3										5					
		0.101	2.25	0.170			0.170	0.076	NG		5					
		0.133	1.00								4					
		0.101	2.25	0.197	1.000	1.000	0.197	0.076	NG							
X		0.206	1.00	[0.154]			0.154	[0.154]	NG		5	10528.902	0.75	1050	1.4	1
	2										5					
		0.080	2.25	0.155			0.155	0.069	NG		5					
		0.095	1.00													
		0.080	2.25	0.110	1.000	1.000	0.110	0.069	NG		4					
		0.153	1.00	[0.131]			0.131	[0.131]	NG		5	14604.606	0.857142857	1200	1.9	2
	1										5					
		0.044	2.25	0.099			0.099	0.044	NG		5					
		0.180	1.00													
		0.044	2.25	0.110	1.000	1.000	0.110	0.044	NG		4					
		0.078	1.00	[0.078]			0.078	[0.078]	NG		5	18680.31	1	1400	2.7	3

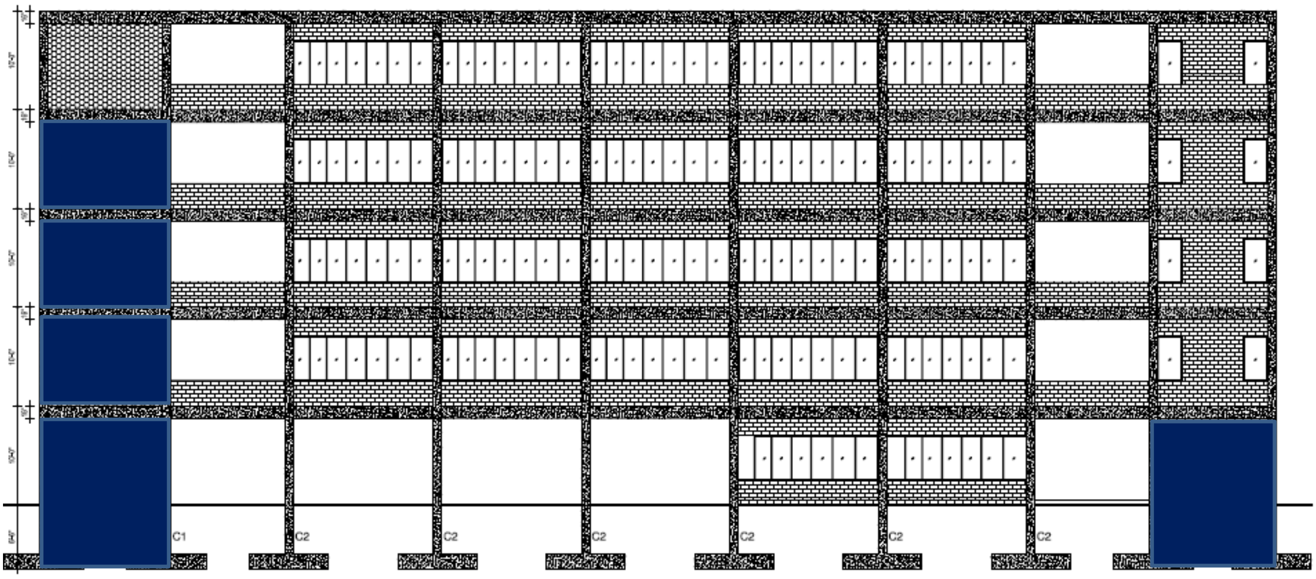
ADDING SW IN X-DIR (GROUND FLOOR)



ADDING SW IN X-DIR (1ST FLOOR,2ND ,3RD FLOOR)

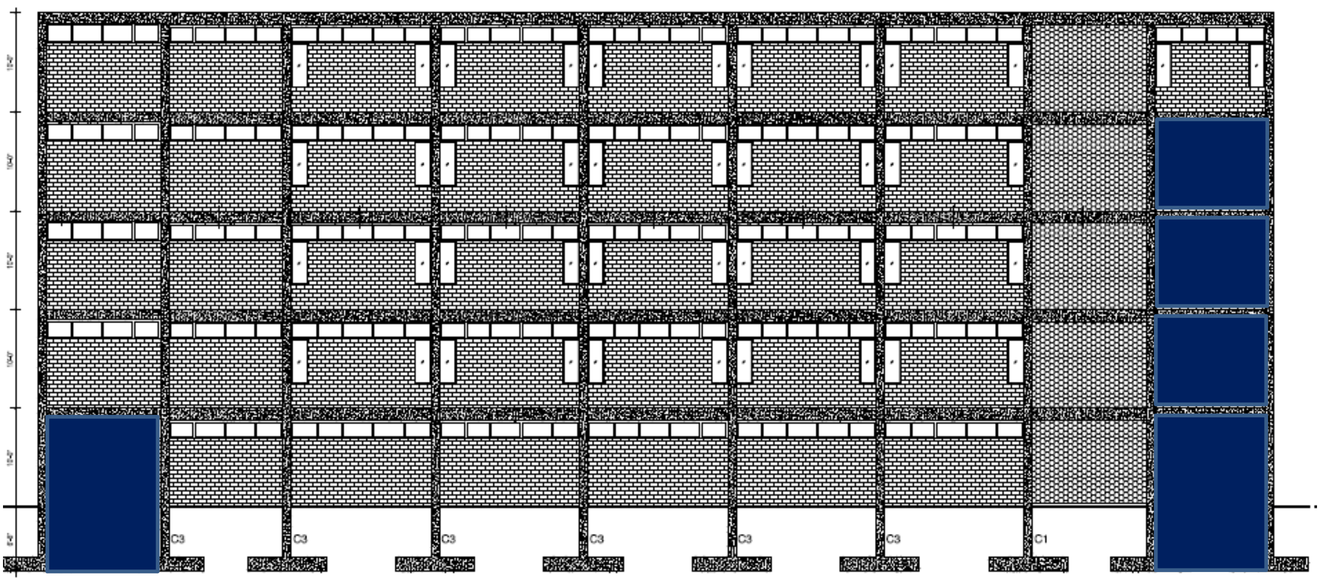


ELEVATION GRID A-A



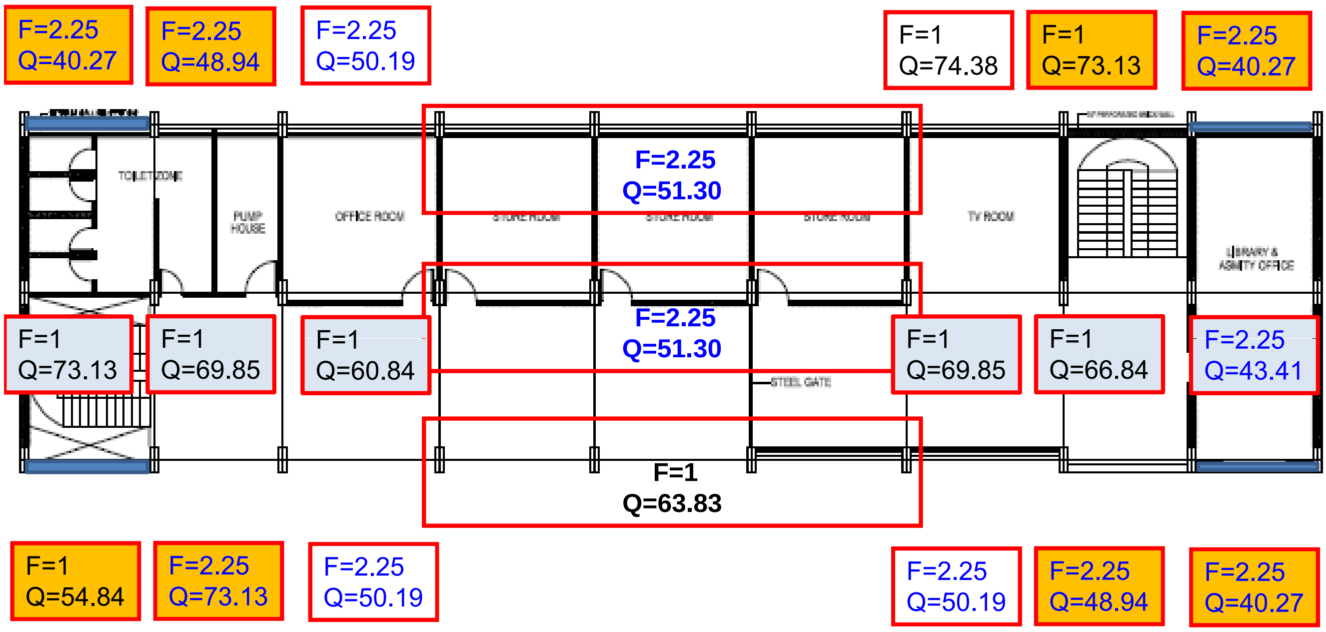
ELEVATION GRID A-A

ELEVATION GRID C-C



ELEVATION GRID C-C

BUILDING ASSESSMENT (X DIRECTION STORY 1 AFTER)



F=1 QT(-)=874.3-(54.84+73.13)+(0.72)(823.1-40.27*3-48.94*2)+4*1400=6781.5

C=6781.5/18692.2=0.36

F=2.25 QT=823.1-40.27*3-48.94*2=604.41KN

C=604.41/18692.2=0.032

Eo CALCULATION (X DIR STORY 1 AFTER INSERTION OF 4-WALL)

STRENGTH DOMINANT STRUCTURE

Σwi= 18692.2 kN

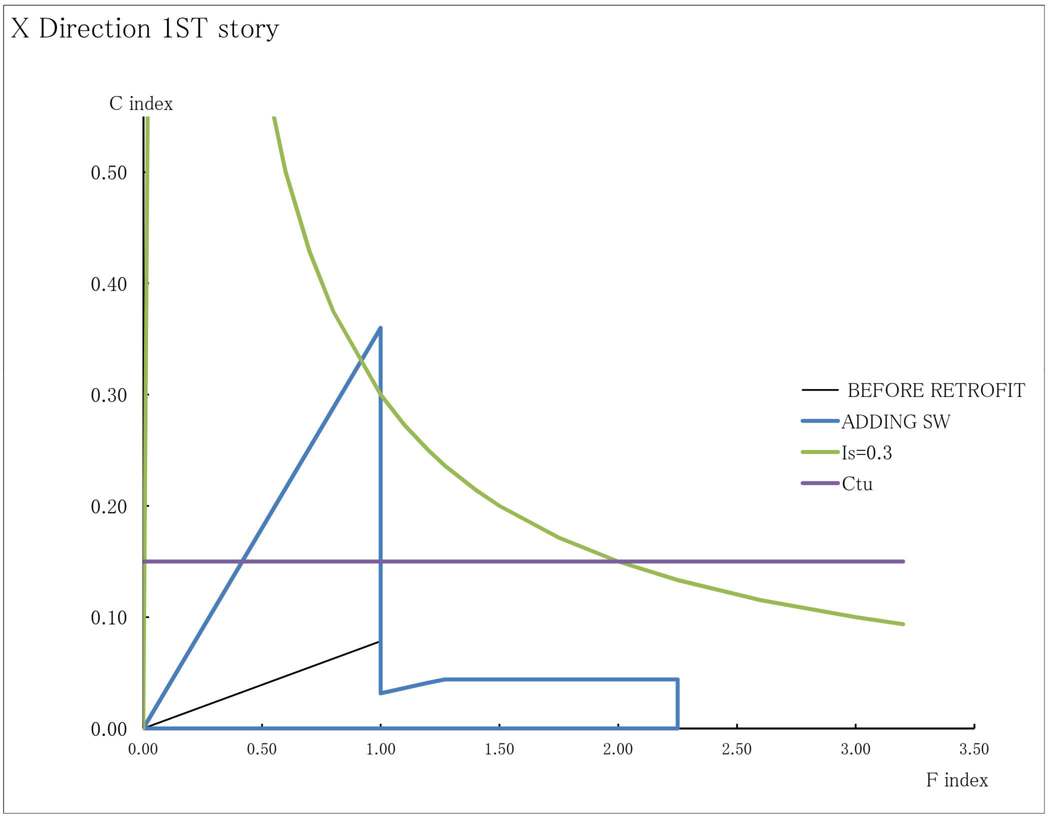
Direction	Story	GN	Q	C	ΣQ	C1	F	E0-1	E0-2	Ctu	
X	1	1	0.0	0.00	0.00	0.000	0.80				
		2	6346.33	0.33	6781.50	0.36	1.00		0.36	0.36	
		3	0.0	0.00	0.00	0.000	1.10				
		4	0.0	0.00	0.00	0.000	1.20				
		5	0.0	0.00	0.00	0.000	1.27				
		6	0.0	0.00	0.00	0.000	1.40				
		7	0.0	0.00	0.00	0.000	1.50				
		8	0.0	0.00	0.00	0.000	1.75				
		9	0.0	0.00	0.00	0.000	2.00				
		10	604.41	0.032	604.41	0.032	2.25		0.072	0.032	
		11	0.0	0.00	0.00	0.000	2.60				
		12	0.0	0.00	0.00	0.000	3.00				
		13	0.0	0.00	0.00	0.000	3.20				
		ΣQ	1697.4		MAX_E0	0.36	1.0		0.36		

DUCTILITY DOMINANT STRUCTURE

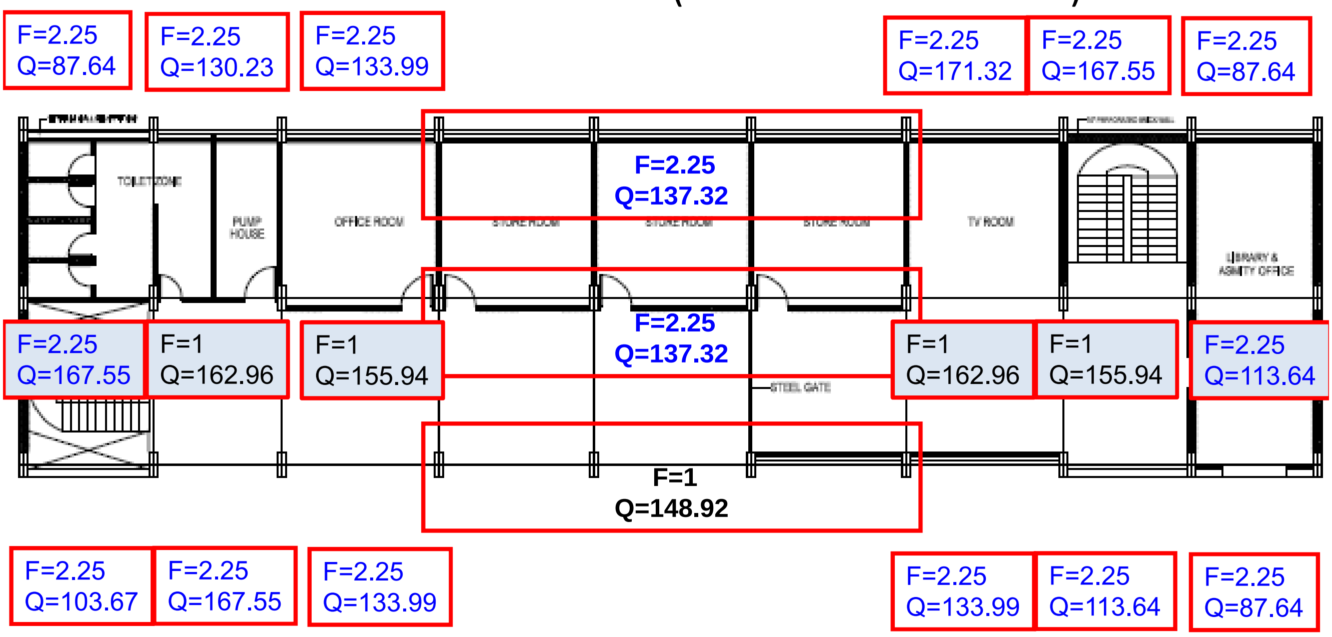
$$E_O = \sqrt{(1*0.36)^2+(2.25*0.032)^2}=0.36$$

PERFORMANCE OF SHEAR WALL

C, F VALUE AFTER ADDING SW IN X DIR 1ST STOREY



BUILDING ASSESSMENT (Y DIRECTION STORY 1)



F=2.25

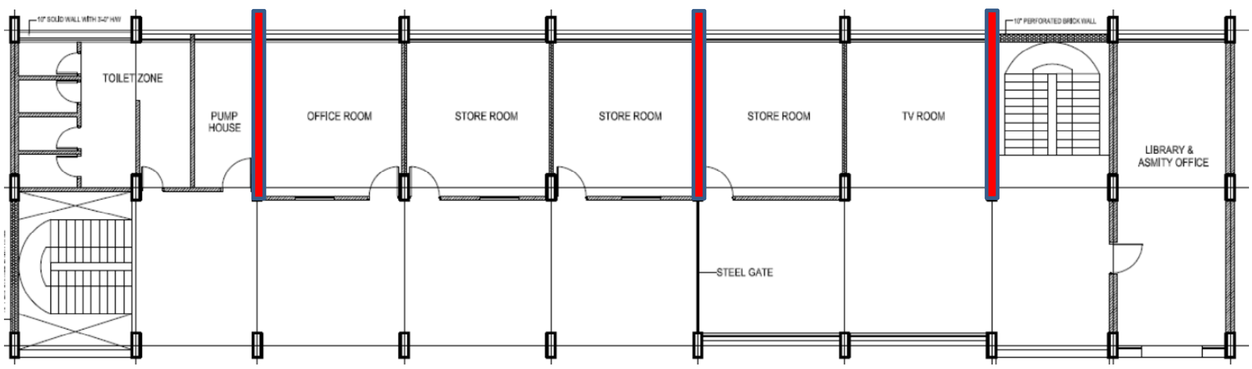
QT=(148.92*4)+(155.94*2)+(162.96*2)=1233.48 KN

C=1233.48/18692.2=0.06

CALCULATION FOR SW(Y-DIR)

Direction	Story	C	F	Eo	T	S_D	Is	$C_{TU} \cdot S_D$	Result	Adoption	Eq	WT (KN)	n+1/(n+i)	SW Cap (KN)	Req SW No=(Is0-Is)XWt/SW cap	No of SW
											5					
		1.789	1.50	1.610			1.610	1.073	OK		5					
	5	0.646	1.50		1.000	1.000										
		1.143	2.25	1.649			1.649	0.686	OK		4					
											5	2377.494	0.6	900	-3.2	Not Required
											5					
		0.746	1.20	0.596			0.596	0.497	OK		5					
	4	0.391	1.20		1.000	1.000										
		0.381	2.25	0.570			0.570	0.254	OK		4					
											5	6453.198	0.67	1000	-1.7	Not Required
											5					
		0.515	1.20	0.463			0.463	0.386	OK		5					
Y	3	0.268	1.20		1.000	1.000										
		0.265	2.25	0.507			0.507	0.198	OK		4					
											5	10528.902	0.75	1125	-1.6	Not Required
											5					
		0.214	2.00	0.367			0.367	0.184	OK		5					
	2	0.189	1.00		1.000	1.000										
		0.214	2.00	0.355			0.355	0.184	OK		4					
		0.343	1.00	[0.294]			0.294	[0.294]	NG		5	14604.606	0.857142857	1285.714286	-0.5	Not Required
											5					
		0.155	2.25	0.349			0.349	0.155	OK		5					
	1	0.066	1.00		1.000	1.000										
		0.155	2.25	0.355			0.355	0.155	OK		4					
		0.178	1.00	[0.178]			0.178	[0.178]	NG		5	18680.31	1	1500	1.6	2

ADDING SW IN Y-DIR (GROUND FLOOR)



CARBON FIBER

$$Q_{su}=\left\{\frac{0.053\cdot p_t^{0.23}\cdot(F_{c1}+18)}{M/(Q\cdot d)+0.12}+0.85\sqrt{p_w\cdot\sigma_{wy}+p_{wf}\cdot\sigma_{fd}}+0.1\sigma_o\right\}\cdot b\cdot j$$

$p_w\cdot\sigma_{wy}+p_{wf}\cdot\sigma_{fd}$ shall be not more than 9.8 N/mm^2.

carbon fibar		
roll	3	
thickness	0.167	mm
tensile strength	3430	N/mm2
Young's modulus	230000	N/mm2
σ fd	1610	N/mm2
Pwf	0.00401	

p_{wf} = Shear reinforcement ratio of carbon fiber sheet (decimal).

E_{fd} = Young's modulus of carbon fiber sheet

ε_{fd} = Effective strain of carbon fiber sheet at shear failure.

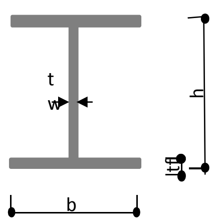
$\sigma_{fd}=\min\{E_{fd}\cdot\varepsilon_{fd},(2/3)\cdot\sigma_f\}$, tensile strength of carbon fiber sheet for shear

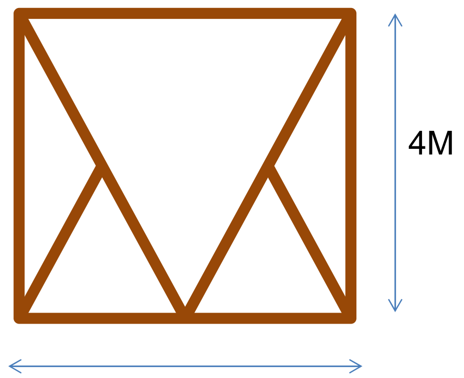
σ_f = Specified tensile strength of carbon fiber sheet.

X DIRECTION:
Qsu=105 KN; F=3.2

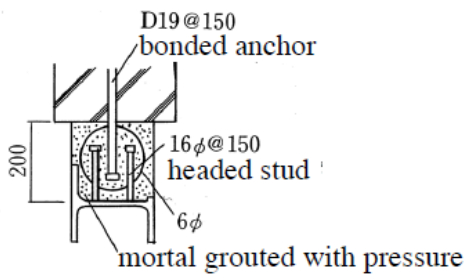
Y DIRECTION:
Qsu=256 KN; F=1

STEEL FRAME





H-250X250X9X14
SS400



X-DIRECTION
Q=2079KN

Y-DIRECTION
Q=1807KN

SELECTION OF METHOD

CHOICE OF METHOD SHOULD CONSIDER

1.USE OF LOCAL AVAILABLE MATERIAL
2.ECONOMY
3.CONSTRUCTION TIME
4.EASE OF CONSTRUCTION
5.QUALITY CONTROL
6.RELABILITY OF METHOD BASED ON TEST DATA
7.ARCHITECTURAL SIMPLICITY & COHERENCE
8.UNINTERREPTED USE DURING RETROFITTING
9.LESS DISTURB THE OCCUPANT
10.MINIMUN MODIFICATION IN PURPOSE OR USE.

THANK YOU

Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings (CNCRP)

Presentation

on

Pushover Analysis for Retrofitting Design

by

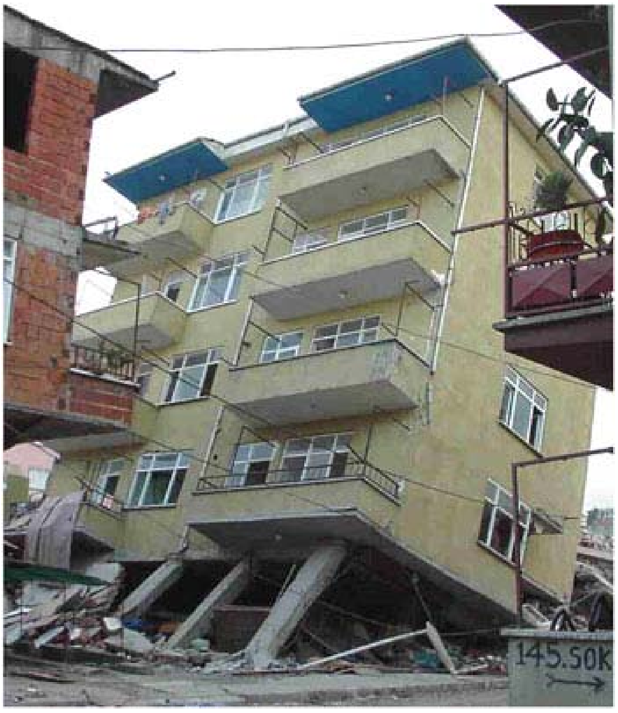
Moniruzzaman Moni

Member, Working Team-2

Introduction

- Nonlinear static analysis or pushover analysis has been developed over the past thirty years
- It is the preferred analysis procedure for design and seismic performance evaluation

Introduction



Introduction

Introduction

Taiwan,1999

Earthquake happens every day somewhere in the world
Earthquake causes loss of human lives and damage of infrastructures

Introduction

80-90% of houses were permanently damaged

Learning from Earthquakes - Sichuan, 2008

Introduction

- Nonlinear Static Procedures (NSP) shall be used for analysis of buildings when linear procedures are not permitted

- The NSP shall be permitted for structures in which higher mode effects are not significant

Key Elements of the Pushover Analysis

- Nonlinear static procedure: constant gravitational loads and monotonically increasing lateral loads
- Plastic mechanisms and P-Δ effects: diplacement or arc length control
- Estimation of the target displacement: elastic or inelastic response spectrum for equivalent SDOF system

Key Elements of the Pushover Analysis

- Lateral load patterns: uniform, modal, ELF force distribution
- Capacity curve: Control node displacement vs base shear force
- Performance evaluation: global and local seismic demands with capacities of performance level

Pushover Modeling (Elements)

- Types of Elements

Truss - yielding and buckling

3D Beam - major direction flexural and shear hinging

3D Column - P-M-M interaction and shear hinging

Panel zone - Shear yielding

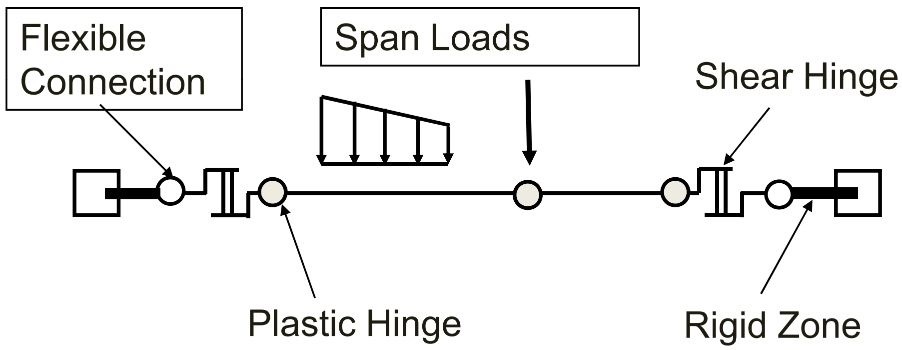
In-fill panel - Shear failure

Shear wall - P-M-Shear interaction

Spring - for foundation modeling

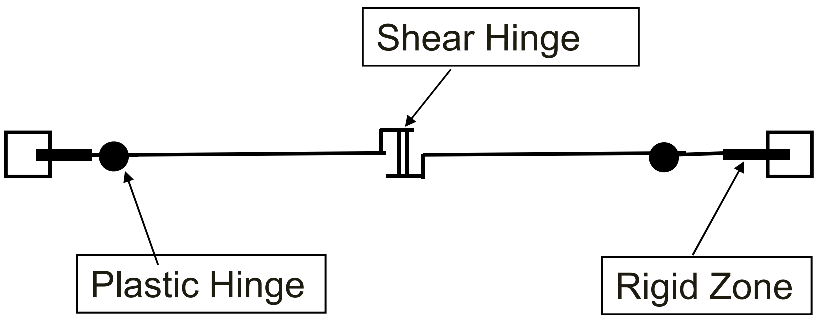
Pushover Modeling (Beam Element)

Three dimensional Beam Element

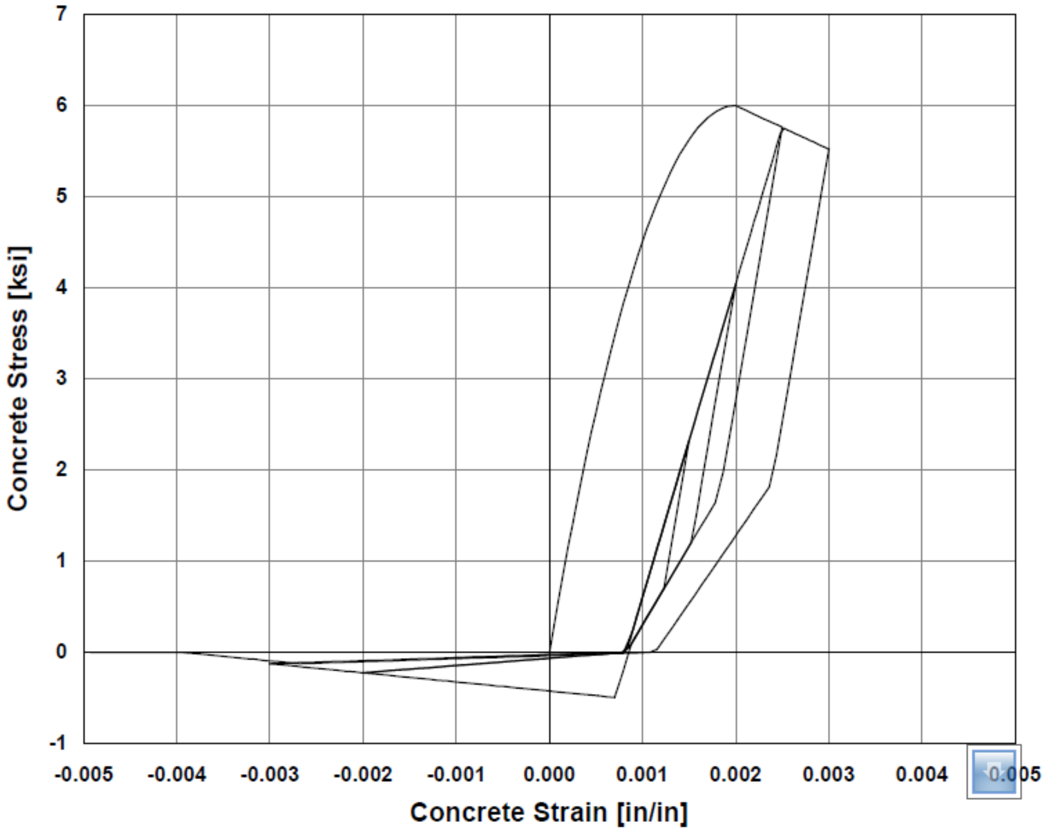


Pushover Modeling (Column Element)

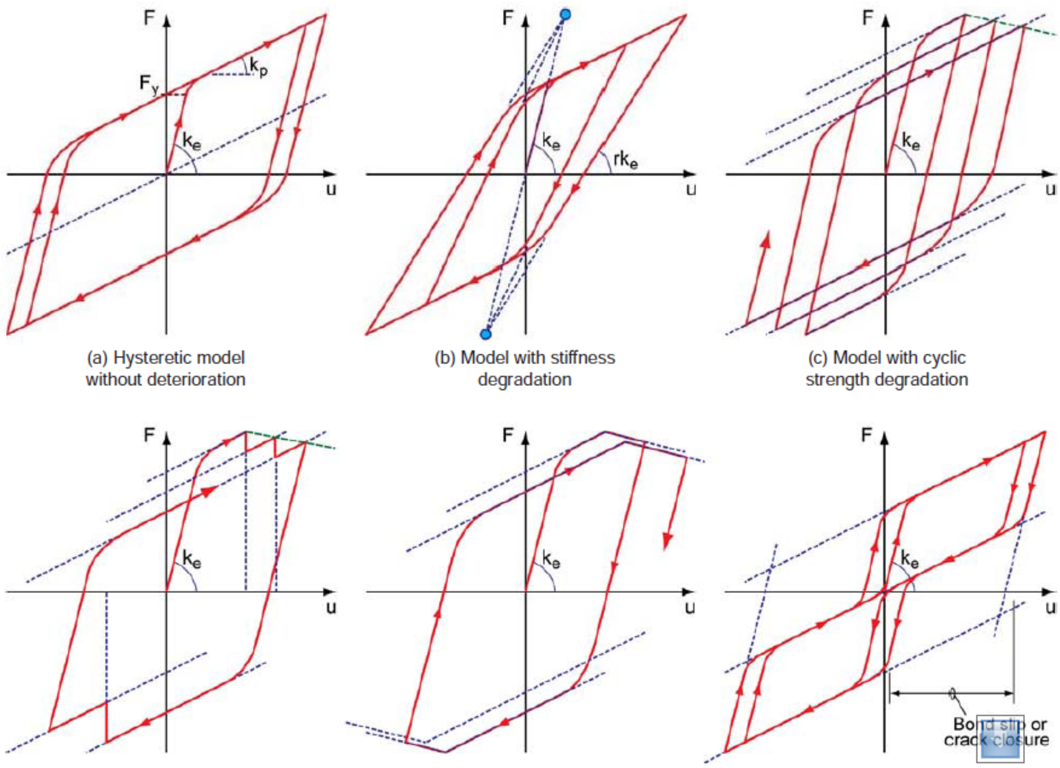
Three dimensional Column Element



Pushover Modeling (Properties)



Pushover Modeling (Properties)



(a) Hysteretic model without deterioration

(b) Model with stiffness degradation

(c) Model with cyclic strength degradation

(d) Model with fracture strength degradation

(e) Model with post-capping gradual strength deterioration

(f) Model with bond slip or crack closure (pinching)

Target Displacement (FEMA-356)

- Estimation of Target Displacement
  - Estimate effective elastic stiffness, K_e
  - Estimate post yield stiffness, K_s
  - Estimate effective fundamental period, T_e
  - Calculate target roof displacement

$$\delta = C_0 C_1 C_2 C_3 S_a T_e^2 /(4\pi^2) * g$$

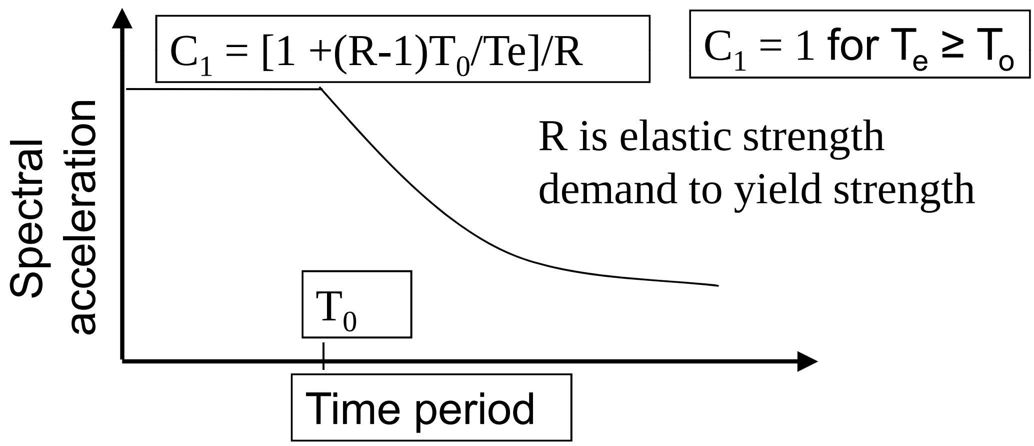
Target Displacement (FEMA-356)

- Calculation of C_0
  - Relates spectral to roof displacement
  - Use modal participation factor for control node from first mode or
  - Use modal participation factor for control node from deflected shape at the target displacement or
  - Use tables based on number of stories and varies from 1 to 1.5

Target Displacement (FEMA-356)

- Calculation of C_1

  Modifier for inelastic displacement



Target Displacement (FEMA-356)

- Calculation of C_2
  - Modifier for hysteresis loop shape
    - Depends on framing type (degrading strength)
    - Depends on performance level
    - Depends on Effective Period
    - 1.0 shall be permitted for nonlinear procedures

Target Displacement (FEMA-356)

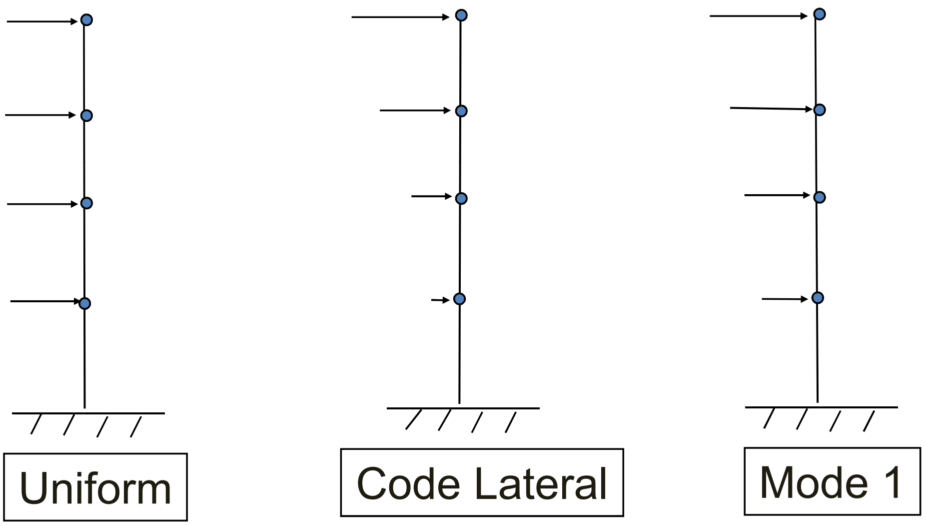
- Calculation of C_3
  - Modifier for dynamic second order effects
    - C_3 = 1 if post yield slope is positive else
    - $C_3 = 1 + [\,|\alpha|(R-1)^{3/2}\,]/T_e$
- R=Ratio of elastic strength demand to calculated yield strength
- S_a = Response spectrum acceleration

Pushover Modeling (Loads)

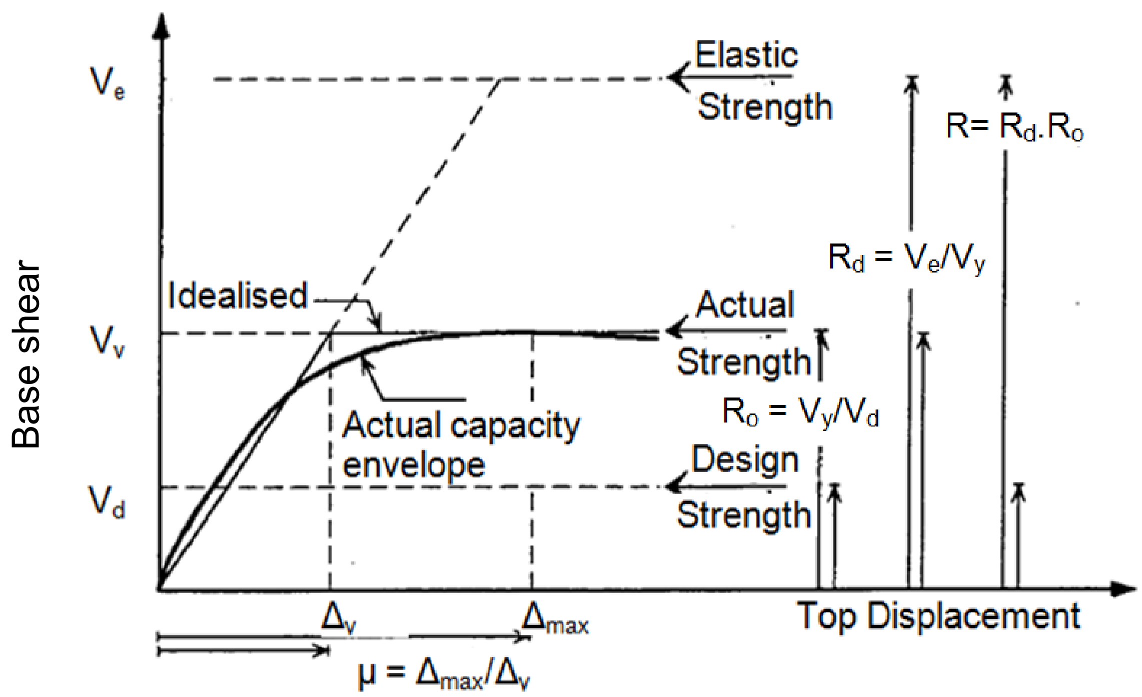
- Start with Gravity Loads
  - Dead Load
  - Some portion of Live Load
- Select Lateral Load Pattern
  - Lateral Load Patterns (Vertical Distribution)
  - Lateral Load Horizontal Distribution
  - Torsional Effects
  - Orthogonal Effects

Pushover Modeling (Loads)

Lateral Load Patterns (Vertical Distribution)

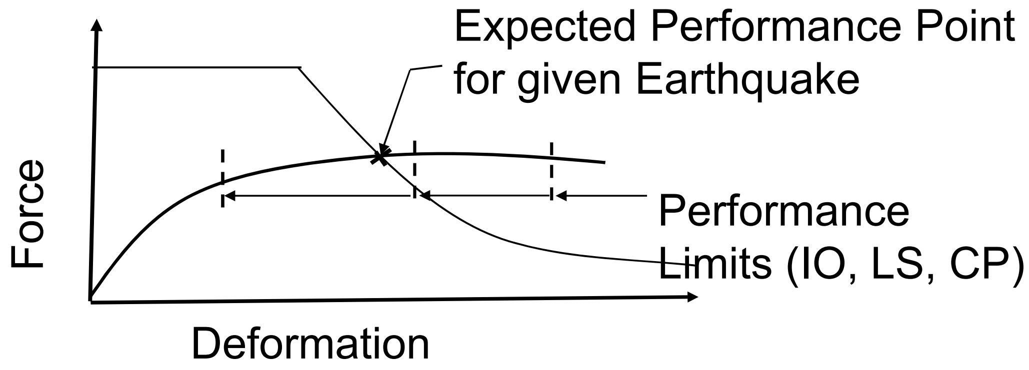


Capacity Curve



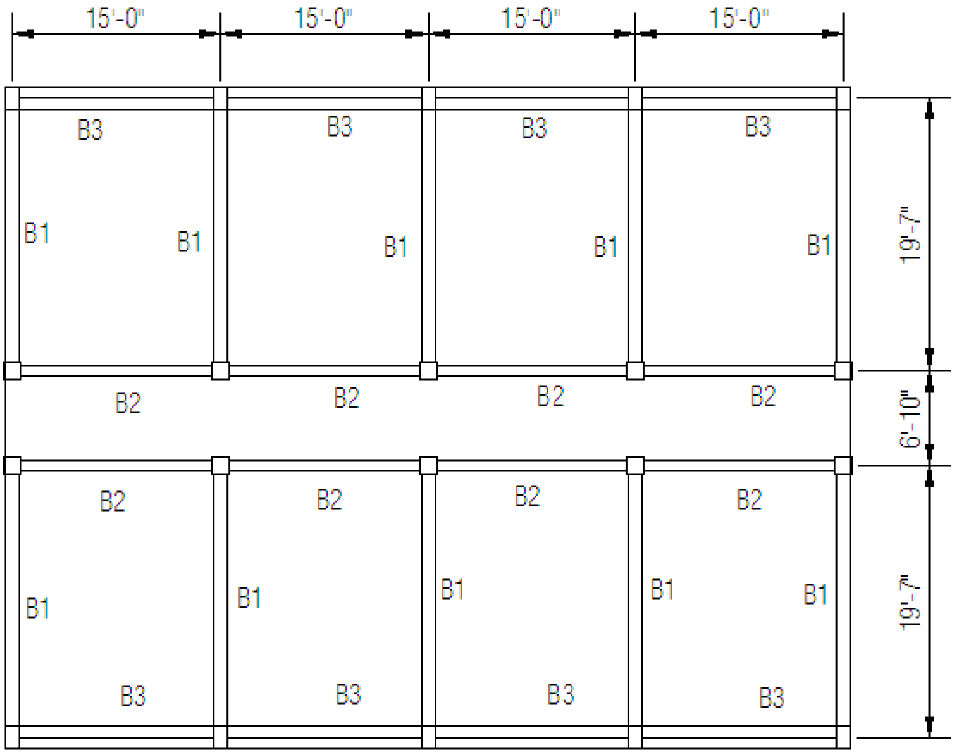
Relation of different factors

Performance Check Using Pushover



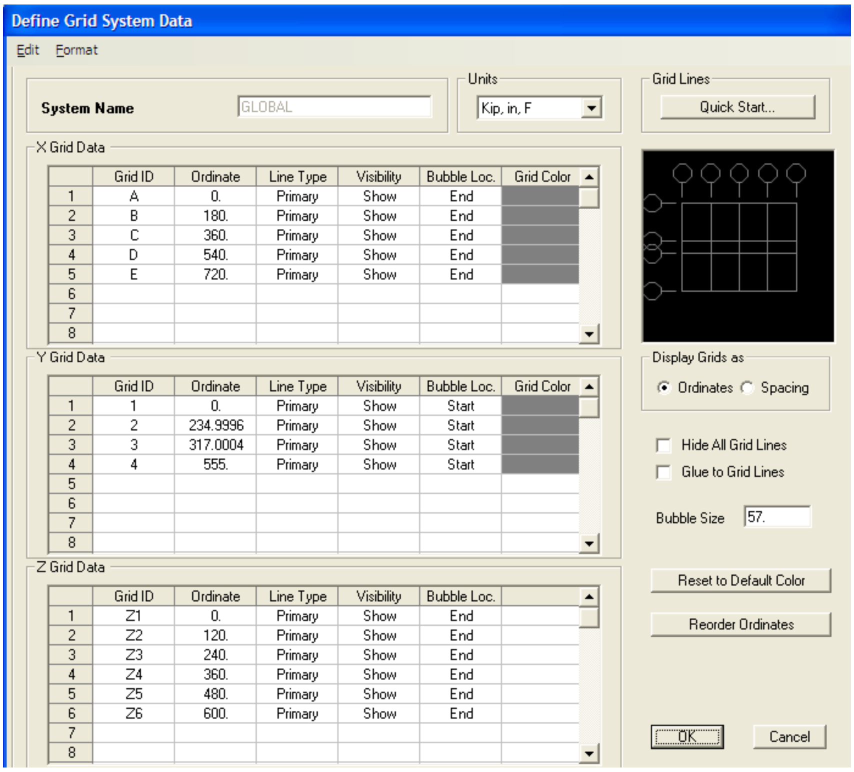
- Construct Pushover curve
- Select earthquake level(s) to check and construct their spectrum curves
- Decide the performance level(s) (i.e.: IO, LS, CP)
- Verify structural performance with guidelines
  - Capacity Spectrum Method (ATC-40)
  - Displacement Coefficient Method (FEMA 356)

Pushover Analysis Example

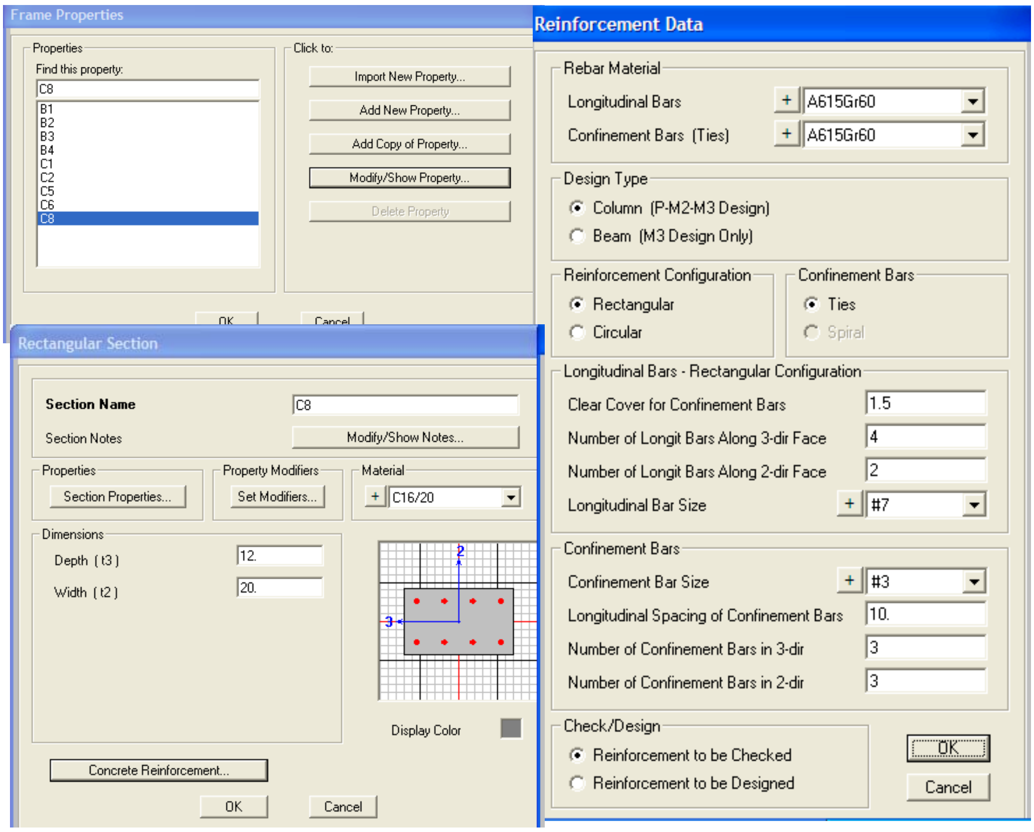


Clinic Building

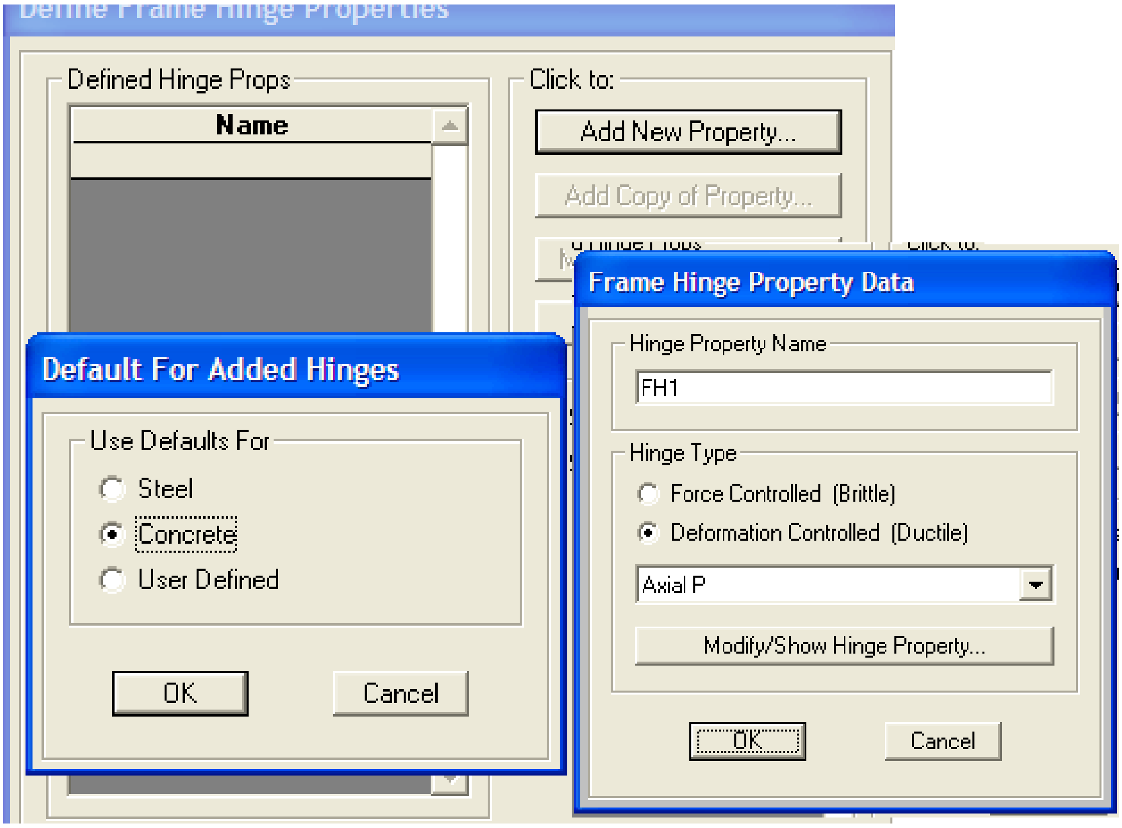
Pushover Analysis Example



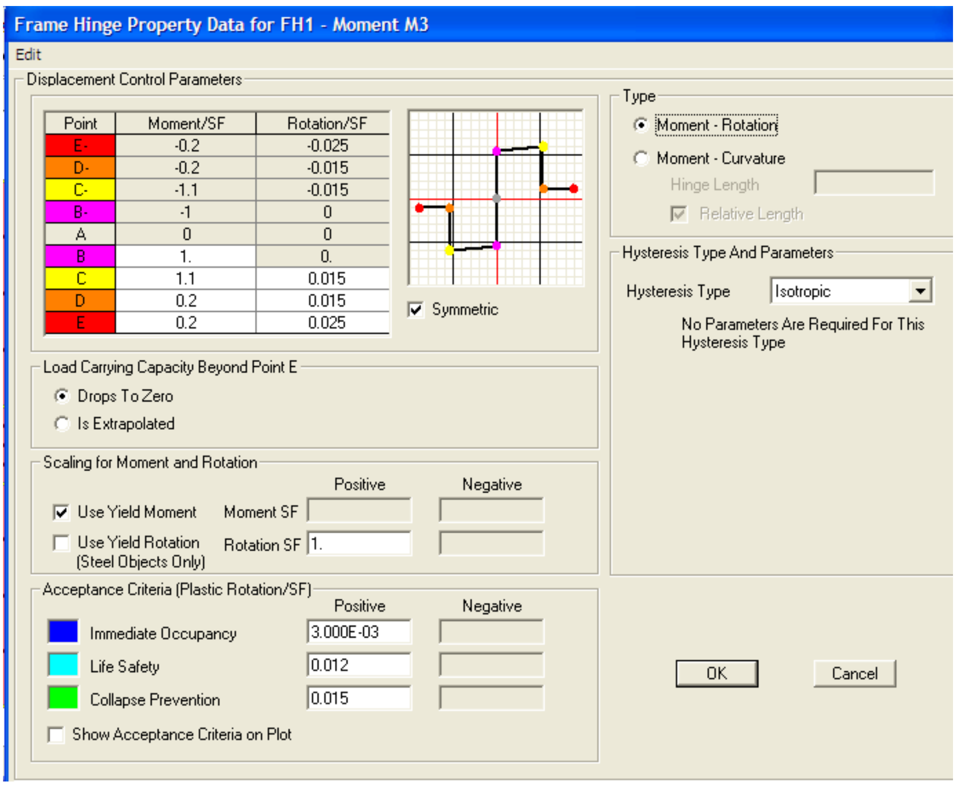
Pushover Analysis Example



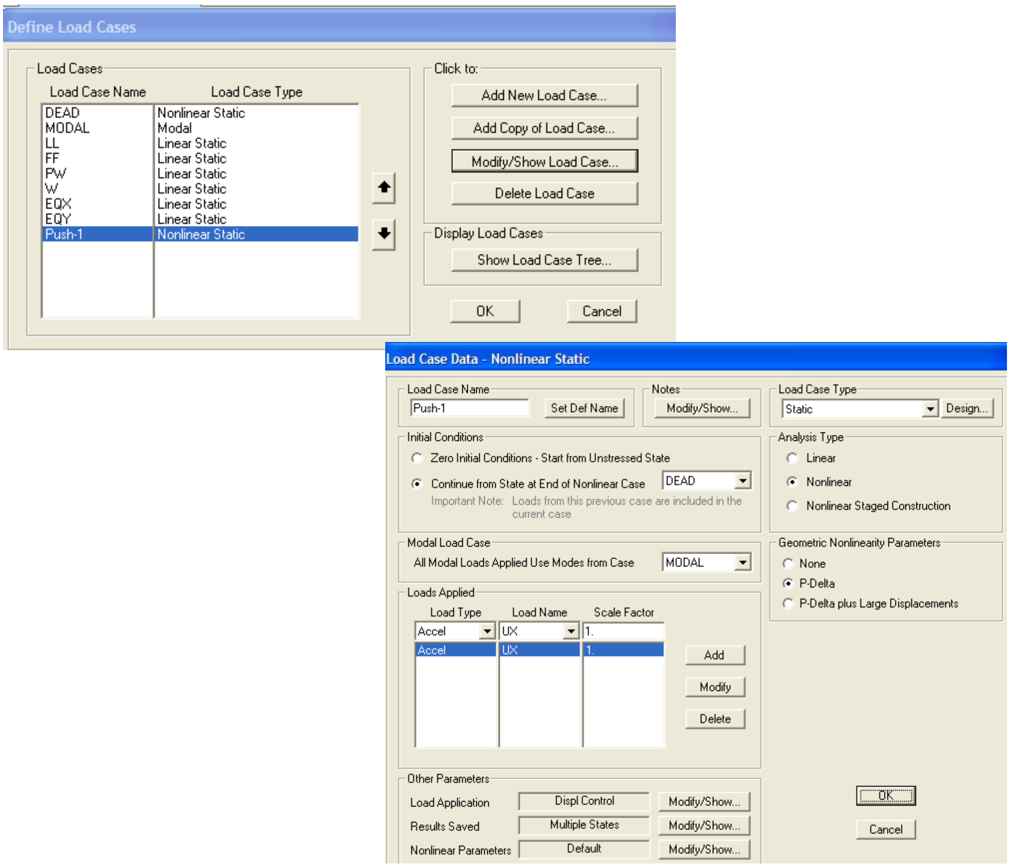
Pushover Analysis Example



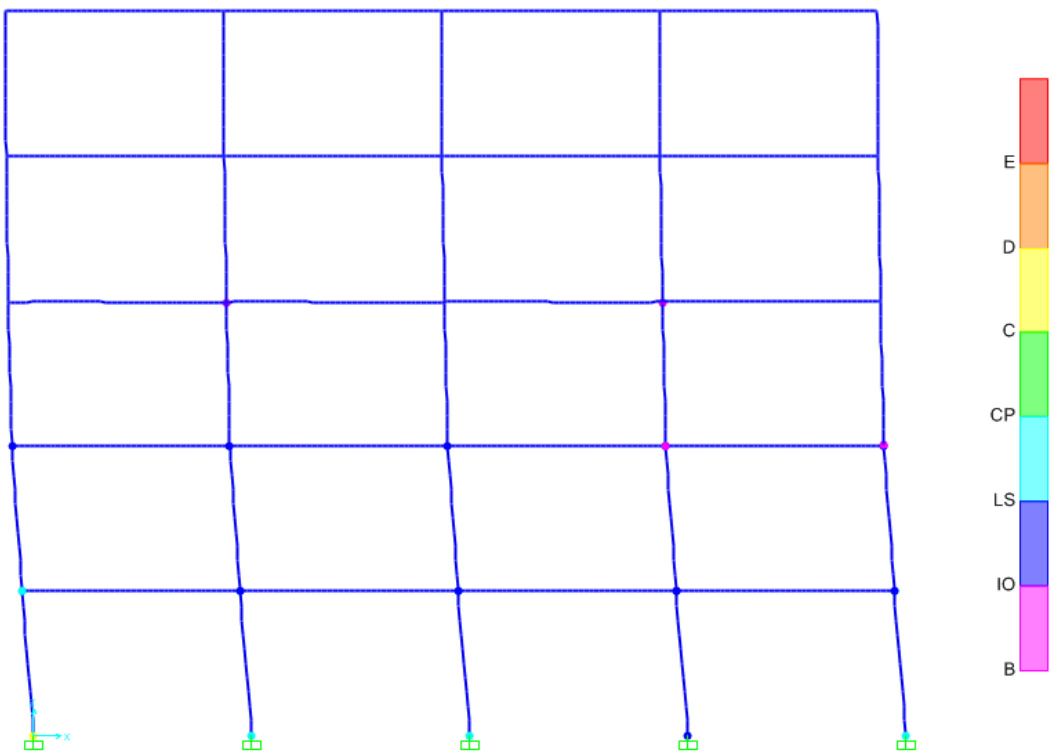
Pushover Analysis Example



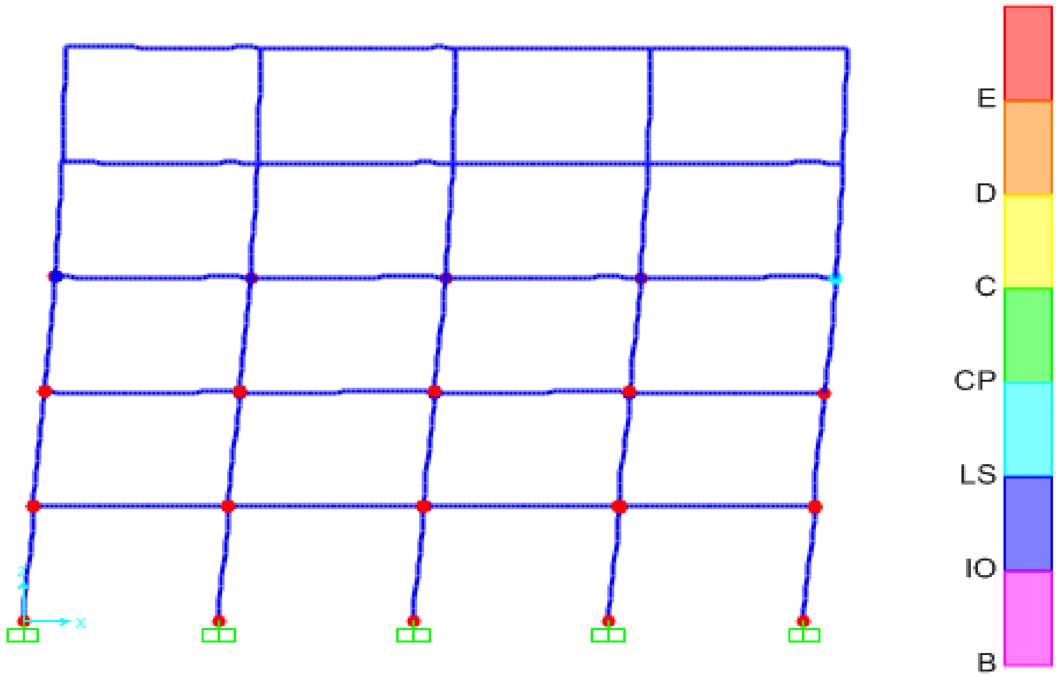
Pushover Analysis Example



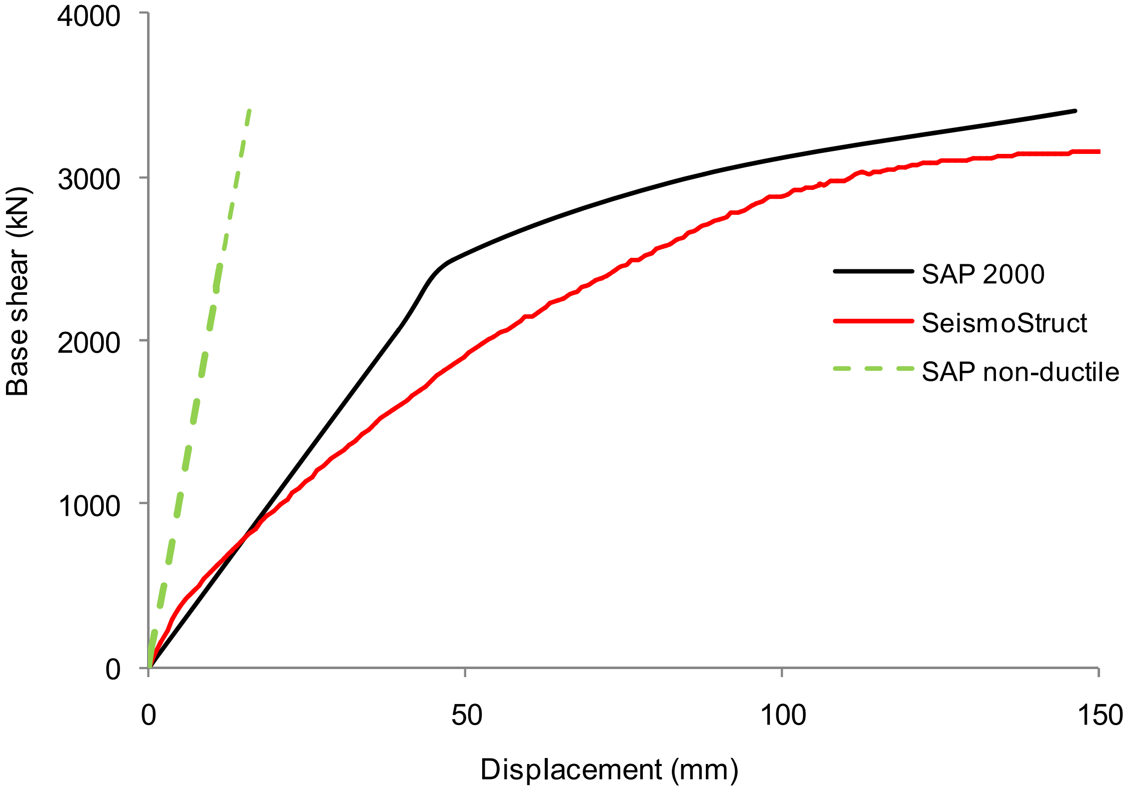
Pushover Analysis (Results)



Pushover Analysis (Results)



Pushover Analysis (Results)



SEISMIC DESIGN CONCEPT FOR NEW BUILDINGS

MD. MOMINUR RAHMAN
EXECUTIVE ENGINEER
PUBLIC WORKS DEPARTMENT AND
TEAM MEMBER, COMPONENT-2
CNCRP PROJECT.

An effective seismic design generally includes

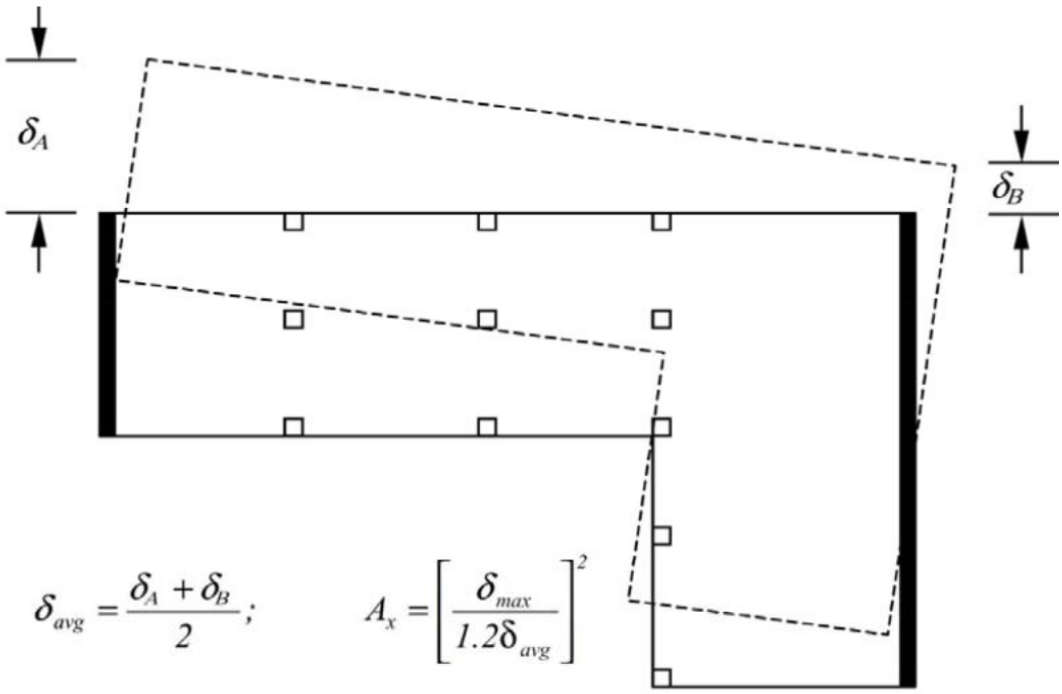
1. Layout of a lateral force-resisting system which includes providing a redundant and continuous load path to ensure that a building responds as a unit during ground motion.

2. Determination of code-prescribed forces and deformations generated by the ground motion, and distribution of the forces vertically to the lateral force-resisting system.

3. Analysis of the building for the combined effects of gravity and seismic loads to verify that adequate vertical and lateral strengths and stiffnesses are achieved to satisfy the structural performance and acceptable deformation levels prescribed in the building code.

4. Structural detailing to assure that the structure has sufficient inelastic deformability to undergo large deformations when subjected to a major earthquake.

HORIZONTAL TORSIONAL MOMENTS



$$\delta_{avg} = \frac{\delta_A + \delta_B}{2}; \qquad A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}}\right]^2$$

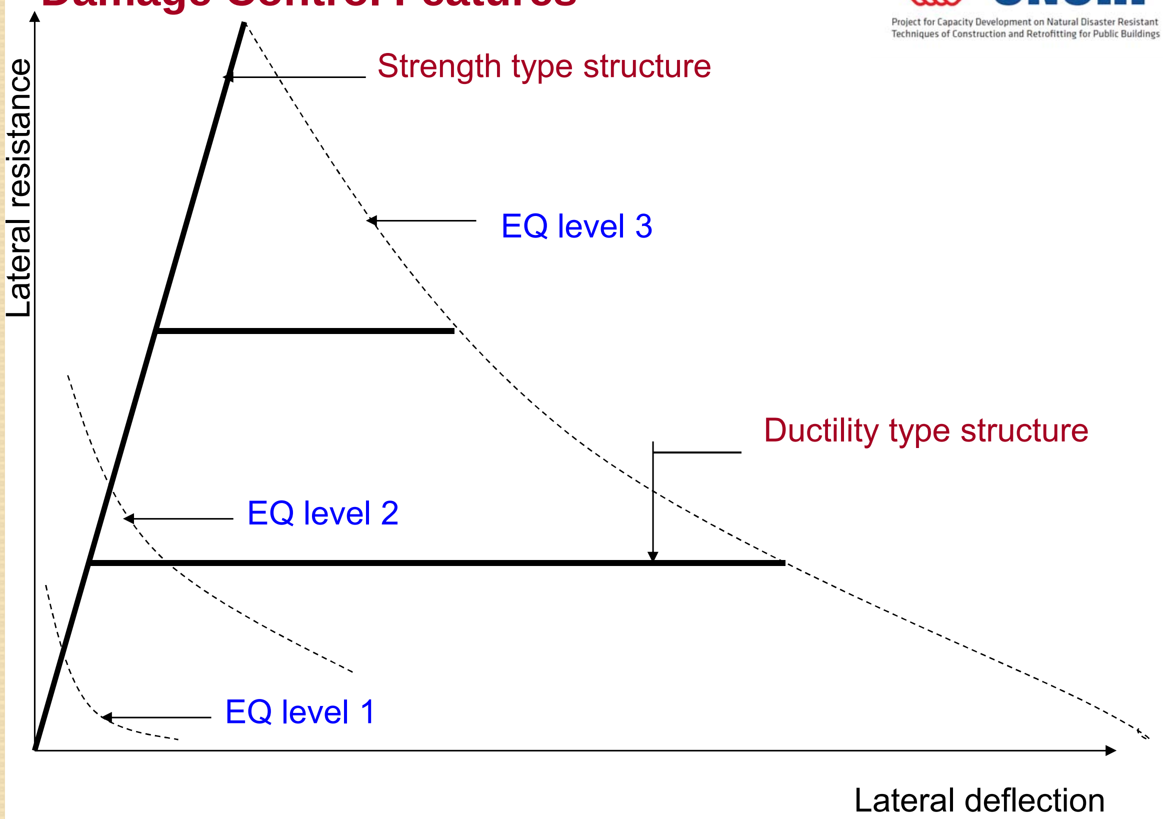
The accidental torsional moment M_{tai} at level *i* is given as:
$M_{tai} = e_{ai} F_i$

where, e_{ai} = accidental eccentricity of floor mass at level i applied in the same direction at all floors = $\pm 0.05\ Li$
L_i = floor dimension perpendicular to the direction of seismic force considered.

Where torsional irregularity exists for Seismic Design Category C or D, the irregularity effects shall be accounted for by increasing the accidental torsion M_{tai} at each level by a torsional amplification factor,

$$A_x = [\delta_{max} / (1.2\delta_{avg})]^2 \leq 3.0$$

Damage Control Features



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To minimize the damage of nonstructural elements, special care in detailing, either to isolate these elements or to accommodate the movement, is required.

Breakage of glass windows can be minimized by providing adequate clearance at edges to allow for frame distortions.

Damage to rigid nonstructural partitions can be largely eliminated by providing a detail at the top and sides, which will permit relative movement between the partitions and the adjacent structural elements.

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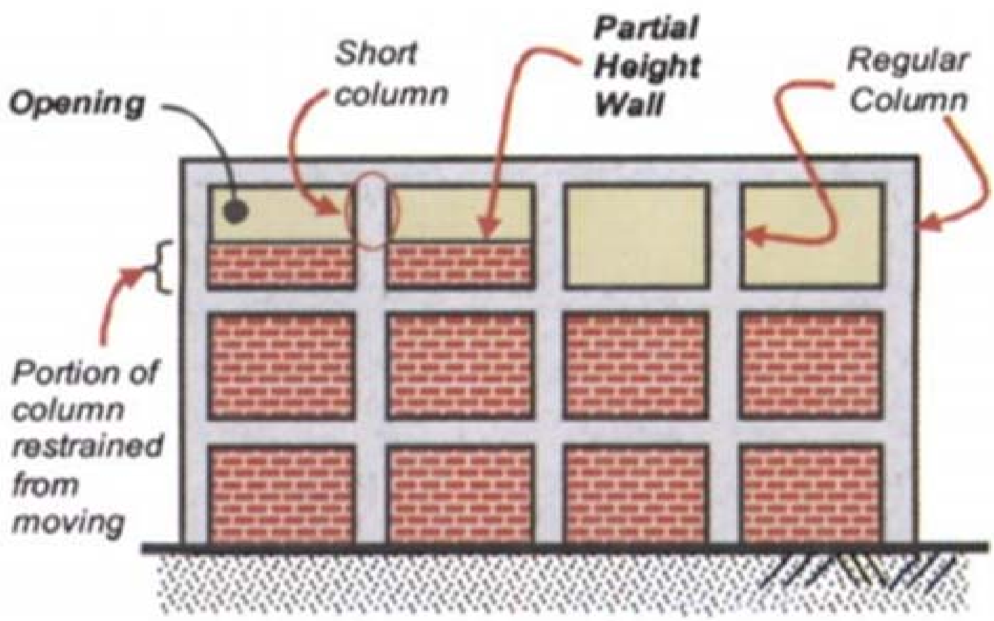
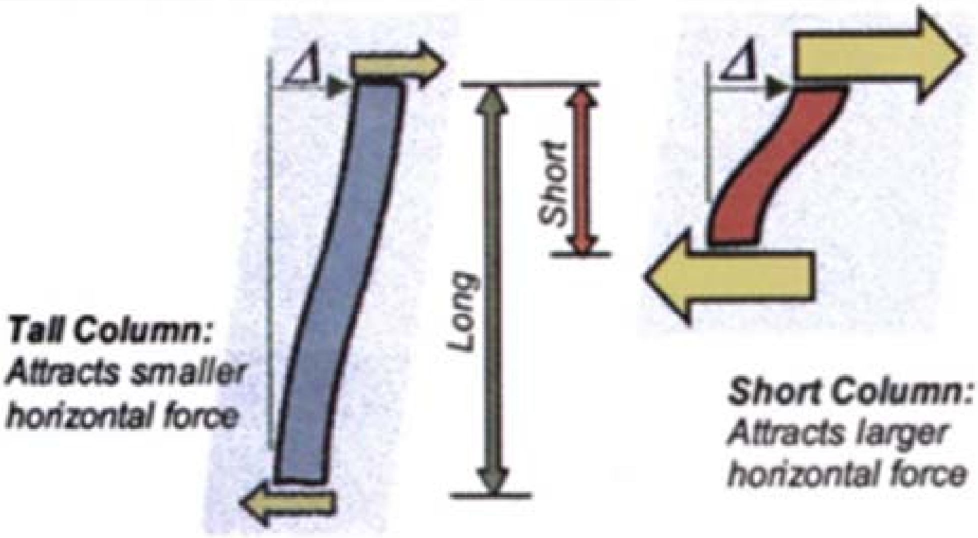
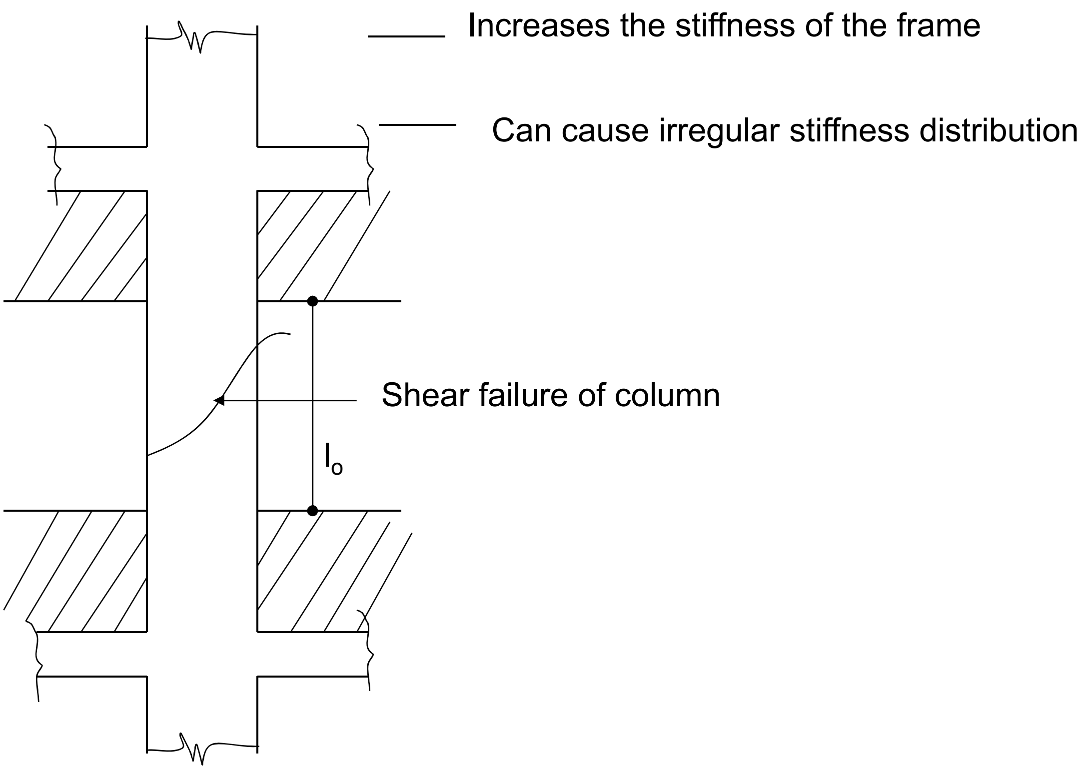
Effect of non-structural elements

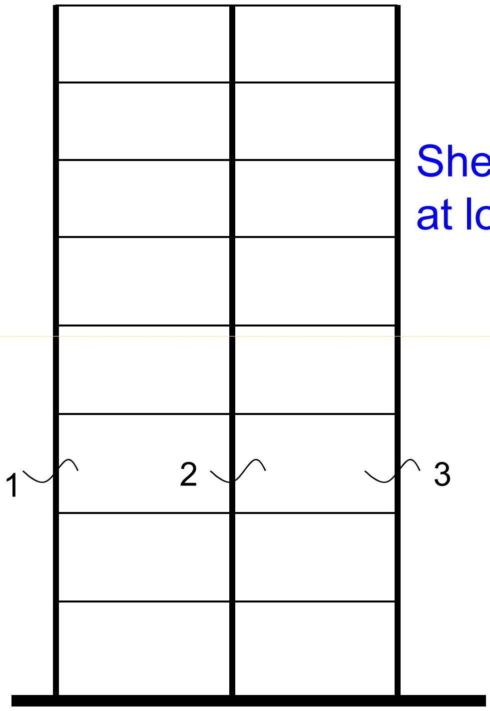




Poor behaviour of short columns is due to the fact that in an earthquake, a tall column and a short column of same cross section move horizontally by same amount which can be seen from the given figure.



Progressive collapse of a brittle structure



Shear failure of short column occurs at low deformation.

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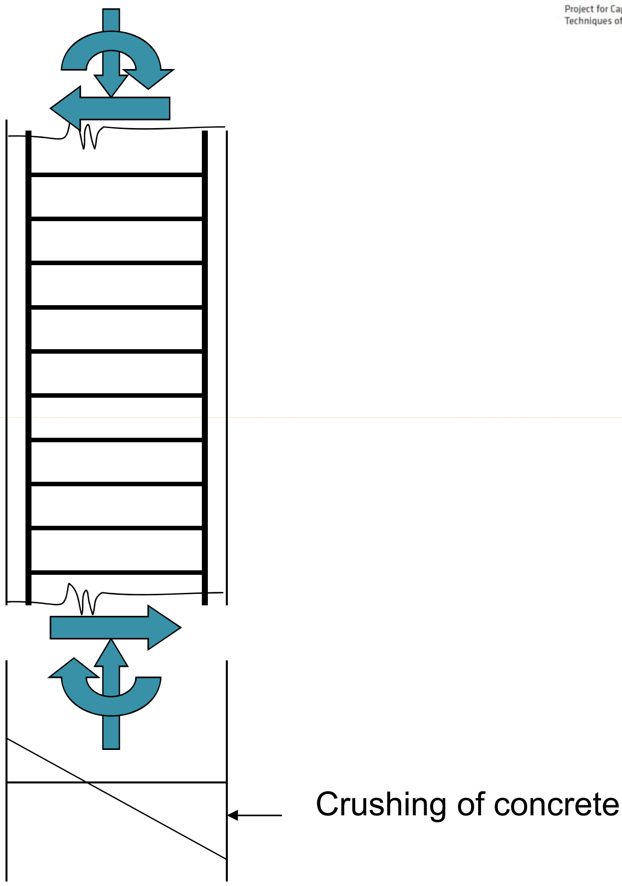
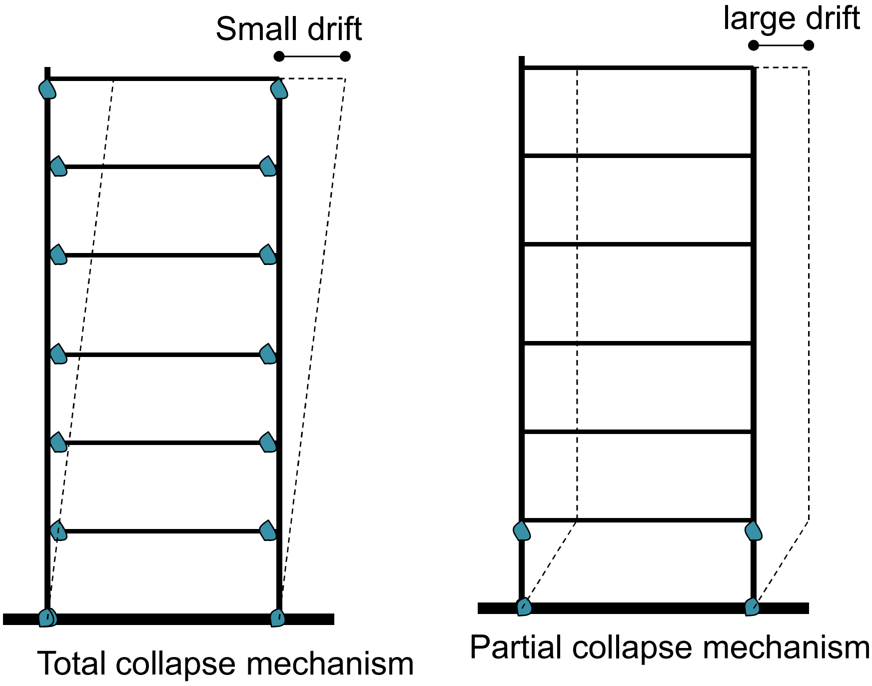
Short column failure

Flexural compression failure of corner colum





Soft story failure

Effect of column tie on shear failure of column

1. If column tie is absent, brittle shear failure occurs in diagonal tension mode.

2. If minimum column tie is present, diagonal compression failure of concrete occurs after tie yielding. Not brittle failure, but deformation capacity is low.

3. Tie resists tension under shear and must be 135^0 hook.

•Pounding effect

•Deterioration with age

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Beam-column joint failure

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Splicing failure of column main reinforcement

Poor quality of concrete

SITE INVESTIGATION

Appropriate site investigations should be carried out to identify the ground conditions influencing the seismic action. The ground conditions at the building site should normally be free from risks of ground rupture, slope instability and permanent settlements caused by liquefaction or densification during an earthquake. The possibility of such phenomena should be investigated in accordance with standard procedures.

Liquefaction potential and possible consequences should be evaluated for design earthquake ground motions consistent with peak ground accelerations. Any settlement due to densification of loose granular soils under design earthquake motion should be studied. The occurrence and consequences of geologic hazards such as slope instability or surface faulting should also be considered. The dynamic lateral earth pressure on basement walls and retaining walls during earthquake ground shaking is to be considered as an earthquake load for use in design load combinations.

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Foundation failure due to liquefaction

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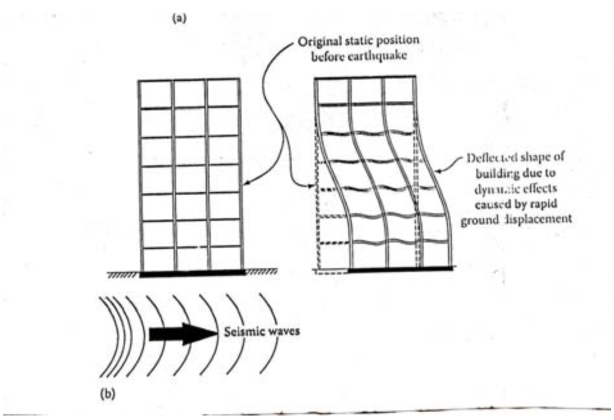
Structural Response

The inertia forces generated by the horizontal components of ground motion require greater consideration for seismic design since adequate resistance to vertical seismic loads is usually provided by the member capacities required for gravity load design.



Load Path

1. There must be a complete gravity and lateral force-resisting system that forms a continuous load path between the foundation and all portions of the building.

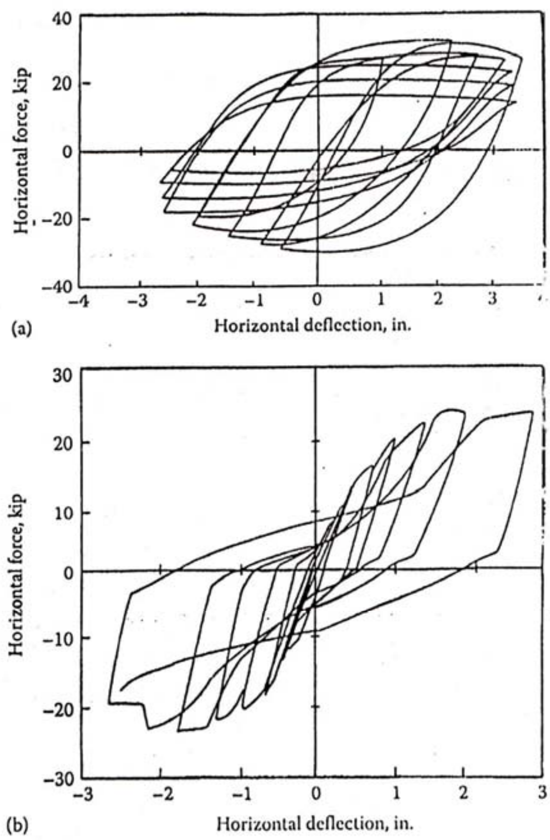
2. If there is a discontinuity in the load path, the building is unable to resist seismic forces regardless of the strength of the elements.

3. Interconnecting the elements needed to complete the load path is necessary to achieve the required seismic performance.

Ductility

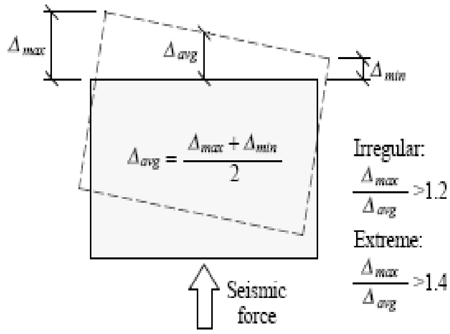
Ductility is the capacity of building materials, systems or structures to absorb energy by deforming into the inelastic range. The capability of a structure to absorb energy, with acceptable deformations and without failure, is a very desirable characteristic in any earthquake-resistant design.

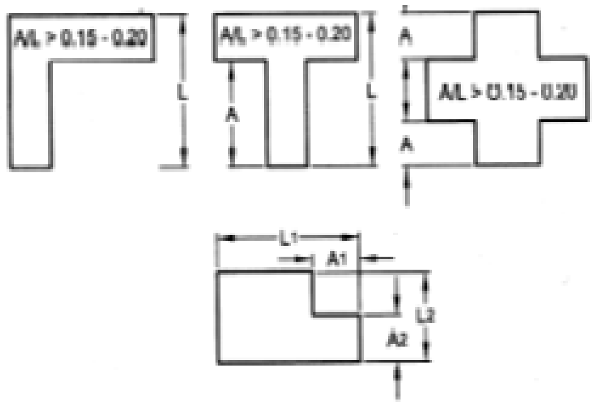
Ductility or hysteretic behavior may be considered as an energy-dissipating mechanism due to inelastic behavior of the structure at large deformations.

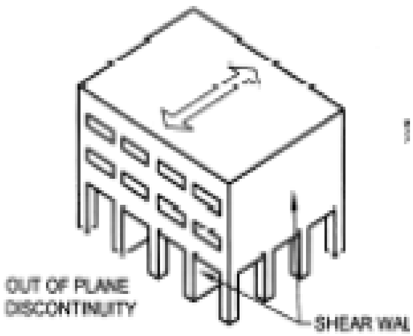


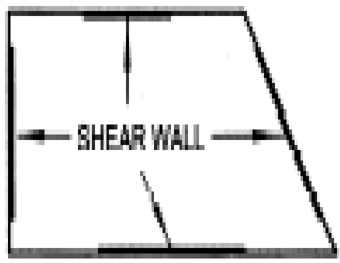
Irregular Buildings

Geometric configuration, type of structural members, details of connections, and materials of construction, all have a profound effect on the structural dynamic response of a building. When a building has irregular features, such as asymmetry in plan or vertical discontinuity, the assumptions used in developing seismic criteria for buildings with regular features may not apply. So it is best to avoid creating buildings with irregular features.









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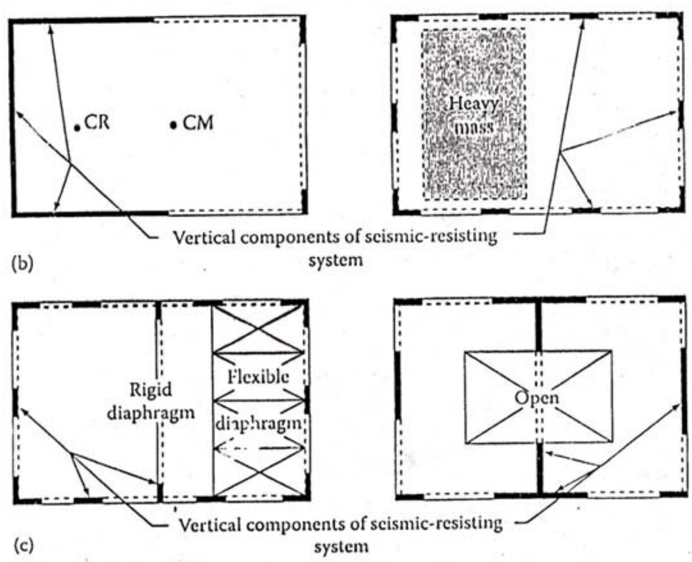
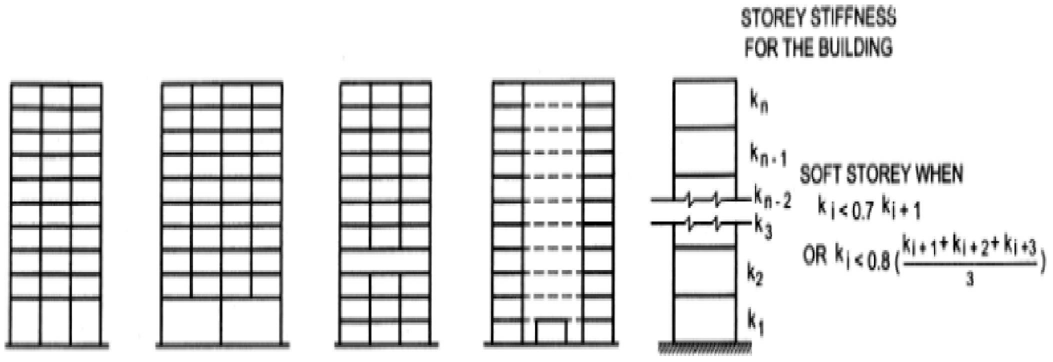
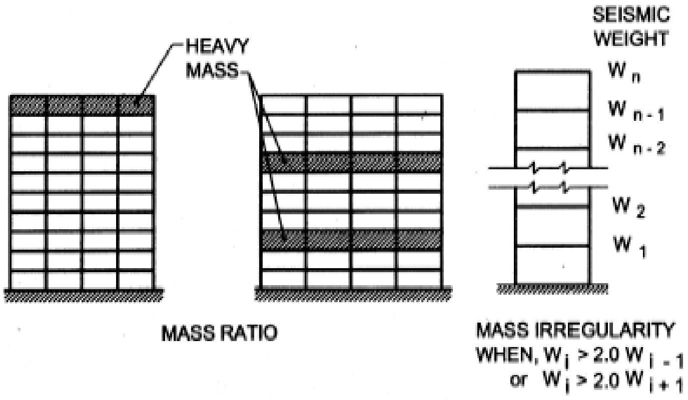
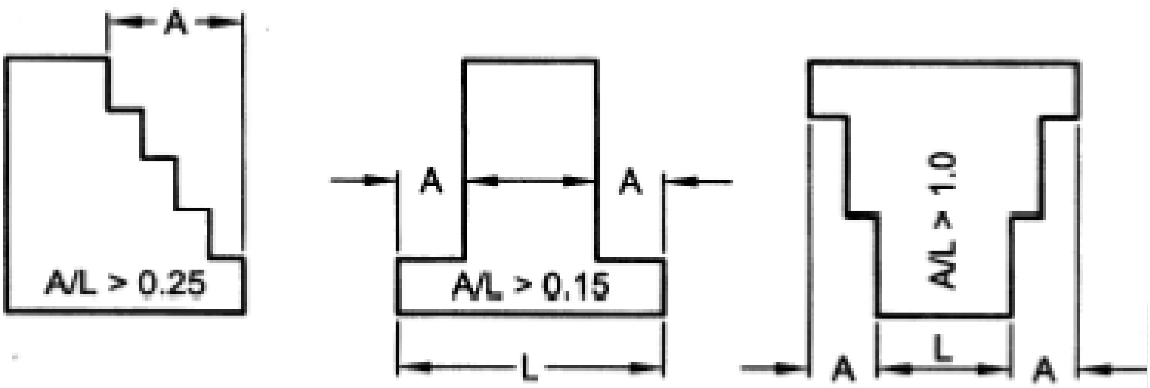
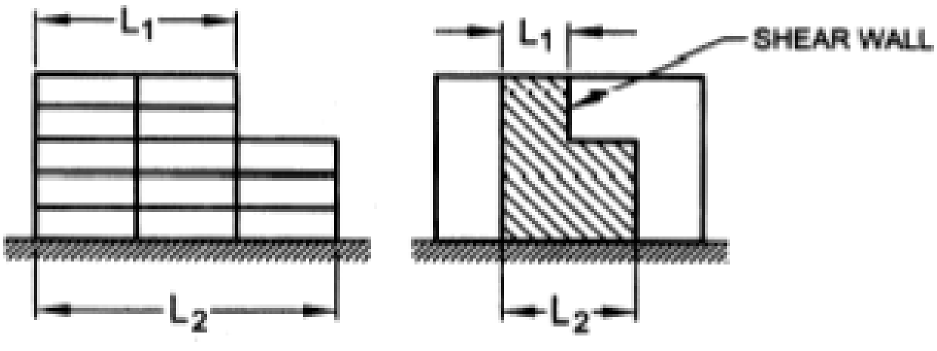
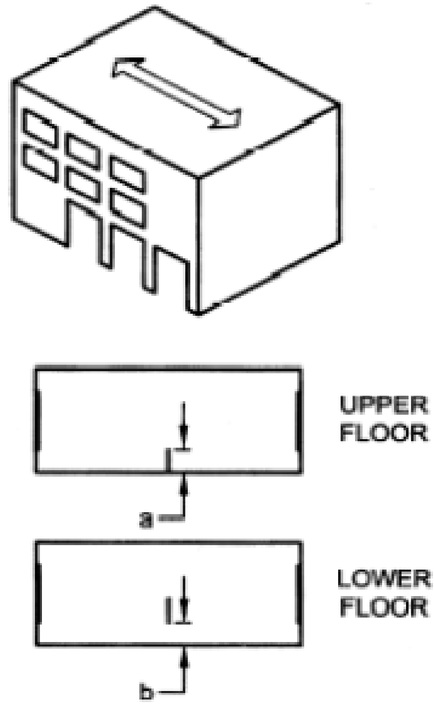



STOREY STIFFNESS
FOR THE BUILDING
kn
kn-1
kn-2
k3
k2
k1
SOFT STOREY WHEN
ki < 0.7 ki+1
OR ki < 0.8 ((ki+1 + ki+2 + ki+3)/3)

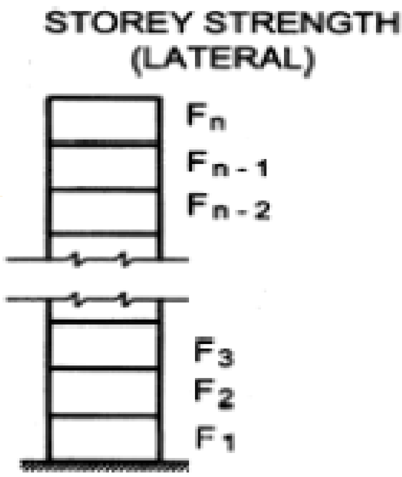

HEAVY
MASS
SEISMIC
WEIGHT
W n
W n-1
W n-2
W 2
W 1
MASS RATIO
MASS IRREGULARITY
WHEN, W i > 2.0 W i-1
or W i > 2.0 W i+1


A
A/L > 0.25
A
A
A/L > 0.15
L
A/L > 1.0
A
L
A


L1
L2
L1
SHEAR WALL
L2



IN - PLANE DISCONTINUITY VERTICAL ELEMENTS RESISTING LATERAL FORCE, WHEN b > a



WEAK STOREY, WHEN $F_i < 0.8 F_i + 1$

Redundancy

A high degree of redundancy accompanied by redistribution capacity through ductility is desirable, enabling a more widely spread energy dissipation across the entire structure and an increased total dissipated energy. The use of evenly distributed structural elements increases redundancy.

The failure of a single connection or component in a building with a redundant system does not adversely affect its lateral stability.

Lateral Force-Resisting Systems

In moment frames, the drift may be large. So a moment-frame building can have substantial nonstructural damage and still be structurally safe.

A shear-wall building is typically more rigid than a framed structure.

Table 2.5.7 Response reduction factor, deflection amplification factor for different Structural Systems and height limitations (m) for different seismic design categories

Seismic Force–Resisting System	Response Reduction Factor, R	Deflection Amplification Factor, C_d	Seis. Design Category B	Seis. Design Category C	Seis. Design Category D
			Height limit (m)		
A. BEARING WALL SYSTEMS (no frame)					
1. Special reinforced concrete shear walls	5	5	NL	NL	50
2. Ordinary reinforced concrete shear walls	4	4	NL	NL	NP
3. Ordinary reinforced masonry shear walls	2	1.75	NL	50	NP
4. Ordinary plain masonry shear walls	1.5	1.25	18	NP	NP
B. BUILDING FRAME SYSTEMS (with bracing or shear wall)					
1. Steel eccentrically braced frames, moment resisting connections at columns away from links	8	4	NL	NL	50
2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	7	4	NL	NL	50
3. Special steel concentrically braced frames	6	5	NL	NL	50
4. Ordinary steel concentrically braced frames	3.25	3.25	NL	NL	11
5. Special reinforced concrete shear walls	6	5	NL	50	50
6. Ordinary reinforced concrete shear walls	5	4.25	NL	NL	NP
7. Ordinary reinforced masonry shear walls	2	2	NL	50	NP
8. Ordinary plain masonry shear walls	1.5	1.25	18	NP	NP

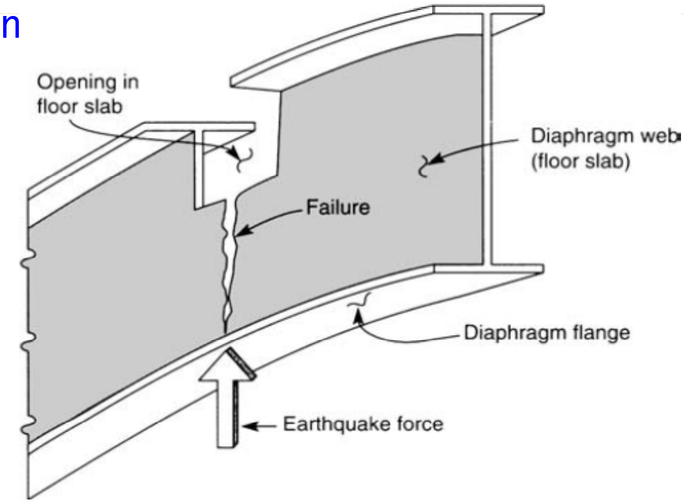
Table 2.5.7 Response reduction factor, deflection amplification factor for different Structural Systems and height limitations (m) for different seismic design categories

Seismic Force–Resisting System	Response Reduction Factor, R	Deflection Amplification Factor, C_d	Seis. Design Category B	Seis. Design Category C	Seis. Design Category D
			Height limit (m)		
C. MOMENT RESISTING FRAME SYSTEMS (no shear wall)					
1. Special steel moment frames	8	5.5	NL	NL	P
2. Intermediate steel moment frames	4.5	4	NL	NL	35
3. Ordinary steel moment frames	3.5	3	NL	NL	NP
4. Special reinforced concrete moment frames	8	5.5	NL	NL	NL
5. Intermediate reinforced concrete moment frames	5	4.5	NL	NL	NP
6. Ordinary reinforced concrete moment frames	3	2.5	NL	NP	NP
D. DUAL SYSTEMS: SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)					
1. Steel eccentrically braced frames	8	4	NL	NL	NL
2. Special steel concentrically braced frames	7	5.5	NL	NL	NL

Seismic Force–Resisting System	Response Reduction Factor, R	Deflection Amplification Factor, C_d	Seis. Design Category B	Seis. Design Category C	Seis. Design Category D
			Height limit (m)		
3. Special reinforced concrete shear walls	7	5.5	NL	NL	NL
4. Ordinary reinforced concrete shear walls	6	5	NL	NL	NP
E. DUAL SYSTEMS: INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)					
1. Special steel concentrically braced frames	6	5	NL	NL	11
2. Special reinforced concrete shear walls	6.5	5	NL	NL	50
3. Ordinary reinforced masonry shear walls	3	3	NL	50	NP
4. Ordinary reinforced concrete shear walls	5.5	4.5	NL	NL	NP
F. DUAL SHEAR WALL-FRAME SYSTEM: ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	4.5	4	NL	NP	NP
G. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE	3	3	NL	NL	NP

Diaphragms

Earthquake loads at any level of a building will be distributed to the lateral load-resisting vertical elements through the floor slabs. Inappropriate location or large size openings for stairs or lift cores create problems similar to those related to cutting the flanges and holes in the web of a steel beam adjacen



Proposed BNBC 2010:
Seismic Design Category: Structural Implications

Seismic design category D has the most stringent seismic design detailing ,while seismic design category B has the least seismic design detailing requirements. Certain structural systems are not permitted for seismic design categories C and D.

Imortance Class I and II					Imortance Class III and IV			
Site class	Zone 1	Zone 2	Zone 3	Zone 4	Zone 1	Zone 2	Zone 3	Zone 4
SA	B	C	C	D	C	D	D	D
SB	B	C	D	D	C	D	D	D
SC	B	C	D	D	C	D	D	D
SD	C	D	D	D	D	D	D	D
SE, S1, S2	D	D	D	D	D	D	D	D

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Dynamic Analysis

For the buildings that are asymmetrical or with areas of discontinuity or irregularity, dynamic analysis is used to determine significant response characteristics such as (a) the effects of the structure's dynamic characteristics on the vertical distribution of lateral forces, (b) the increase in dynamic loads due to torsional motions, (c) the influence of higher modes, resulting in an increase in story shears and deformations.

Static methods specified in building codes are based on single-mode response with simple corrections for including higher mode effects. While appropriate for simple regular structures, the simplified procedures do not take into account the full range of seismic behavior of complex structures. Therefore dynamic analysis is the preferred method for the design of buildings with unusual or irregular geometry.

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REQUIREMENT FOR DYNAMIC ANALYSIS

Dynamic analysis should be performed to obtain the design seismic force, and its distribution to different levels along the height of the building and to the various lateral load resisting elements, for the following buildings:

a) Regular buildings with height greater than 40 m in Zones 2, 3, 4 and greater than 90 m in Zone 1.

b) Irregular buildings with height greater than 12 m in Zones 2,3, 4 and greater than 40 m in Zone 1. For irregular buildings, smaller than 40 m in height in Zone 1, dynamic analysis, even though not mandatory, is recommended.

P-DELTA EFFECTS:

The P - delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered if the stability coefficient (θ) determined by the following equation is not more than 0.10:

$$\theta = P_x \Delta / (V_x h_{sx} C_d)$$

Where,

Px = the total vertical design load at and above level x; where computing *Px*, no individual load factor need exceed 1.0

Δ = the design story drift occurring simultaneously with *Vx*

Vx = the storey shear force acting between levels *x* and *x* − 1

hsx = the story height below level *x*

Cd = the deflection amplification factor given in BNBC

The stability coefficient (θ) shall not exceed θmax.

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THANK YOU

Code Provisions for Seismic Analysis and Design - Example

[Short Training Course on Seismic Assessment, Retrofit Design and Construction of RC Buildings
11-20 Feb, 2013]

Md. Rafiqul Islam
Executive Engineer
PWD Design Division – 3
&
Team Leader
Working Team – 2
CNCRP Project

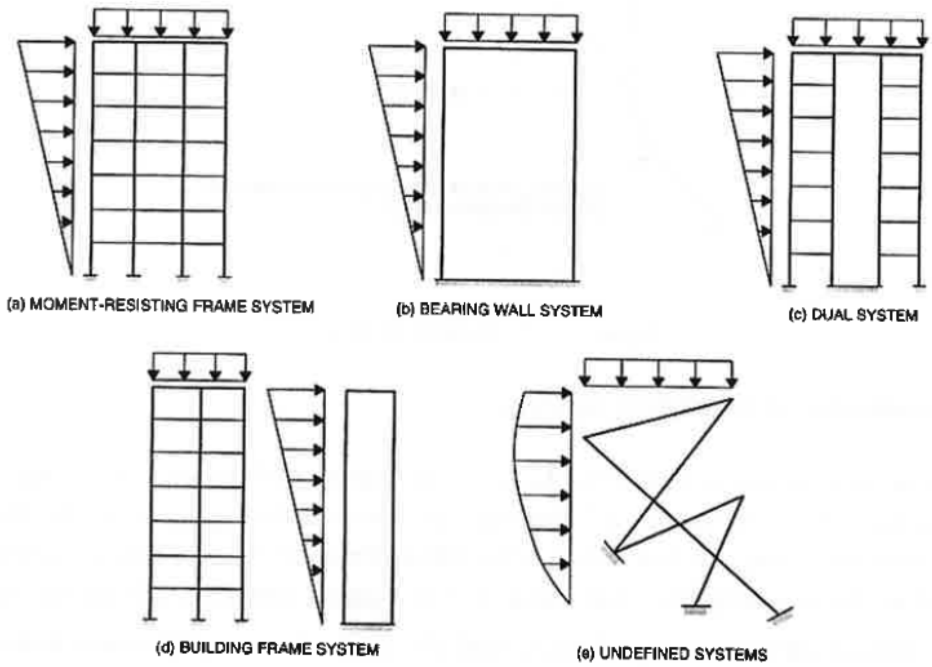
Earthquake Design Philosophy

1. It is uneconomical and unnecessary to design a structure in elastic range for maximum EQ induced inertia force.
2. The large deformation during EQ will be accompanied by yielding in some of the members of the structure.
3. Critical regions of certain members should have sufficient inelastic deformability to dissipate seismic energy.
4. Structure will not collapse when subjected to several cycles of loading into inelastic range.
5. Proper rebar detailing should avoid all forms of brittle failure.

Considerations for EQ Analysis

1. Selection of lateral force resisting system.
2. Check irregularities of structure.
3. Occupancy type of structure.
4. Location of structure in seismic zoning map.
5. Subsoil characteristics

Seismic Force Resisting Structural System



Reference: 'Seismic and Wind Design of Concrete Buildings – S. K. Gosh and Qiang Shen

Types of Moment Frame

Moment Frame: A frame in which member and joint resist lateral forces by flexure.

- Ordinary Moment Frame
- Intermediate Moment Frame
- Special Moment Frame

Ductility is the capacity of building material, systems or structure to absorb energy by deforming into inelastic range.

Choice of Frame (or SDC)

- Restriction from Code
  - Location of building
  - Occupancy type
  - Height of building
  - Soil type
- Choice of the client or designer

✓ Designer must confirm all the provisions of Code of specific frame type.

✓ Site engineer must ensure design and detailing provided by the designer.

Calculation of EQ force

Design base shear $V = S_a W$

S_a = Lateral seismic force coefficient
W = Total seismic weight of the building

In addition to total dead load, consideration for live load are:

a) Live load ≤ 3.0 KN/m^2, consider minimum 25% of live load
b) Live load ≥ 3.0 KN/m^2, consider minimum 50% of live load
c) 100% of permanent heavy equipment or retained liquid or any imposed load

Building Codes Implied Performance

	Return Period
• Ability to resist frequent, minor earthquakes without damage	100 yrs
• Ability to resist infrequent, moderate earthquakes with limited structural and nonstructural damage	475 yrs
• Ability to resist worst earthquakes ever likely to occur without collapse or major life safety endangerment	2475 yrs

Basic consideration:
Design Basis Earthquake (DBE) ground motion = 2/3 of Maximum Considered Earthquake (MCE) ground motion

Design Spectral Acceleration

$$S_a = \frac{2}{3}\frac{ZI}{R}C_s \leq \frac{2}{3}ZI\beta$$

Z = Seismic zone coefficient

I = Structure importance actor

R = Response reduction factor

β = Coefficient for lower bound of S_a = 0.2

C_s = Normalized acceleration response spectrum (function of structure period and soil type)

$$\frac{I}{R} \leq 1.0$$

Site Classification

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity $\bar{V}_s$ (m/s)	Standard Penetration Value, $\bar{N}$ (blows/30cm)	Undrained shear strength, $\bar{S}_u$ (kPa)
SA	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	--	--
SB	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
SC	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
SD	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70

Site Classification

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity $\bar{V}_s$ (m/s)	Standard Penetration Value, $\bar{N}$ (blows/30cm)	Undrained shear strength, $\bar{S}_u$ (kPa)
SE	A soil profile consisting of a surface alluvium layer with V_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $V_s > 800$ m/s.	--	--	--
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	--	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types SA to SE or S_1	--	--	--

Normalized Acceleration Response Spectrum (C_s)

$$C_s = S\left(1+\frac{T}{T_B}(2.5\eta-1)\right) \quad \text{for} \quad 0 \le T \le T_B$$

$$C_s = 2.5S\eta \quad \text{for} \quad T_B \le T \le T_C$$

$$C_s = 2.5S\eta\left(\frac{T_C}{T}\right) \quad \text{for} \quad T_C \le T \le T_D$$

$$C_s = 2.5S\eta\left(\frac{T_C T_D}{T^2}\right) \quad \text{for} \quad T_D \le T \le 4\,\text{sec}$$

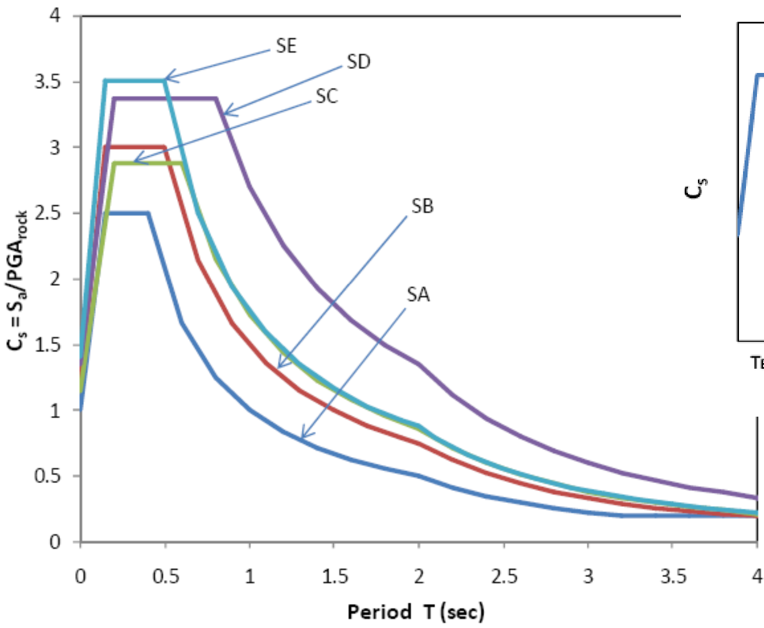
Soil type	*S*	T_B(s)	T_C (s)	T_D (s)
SA	1.0	0.15	0.40	2.0
SB	1.2	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.4	0.15	0.50	2.0

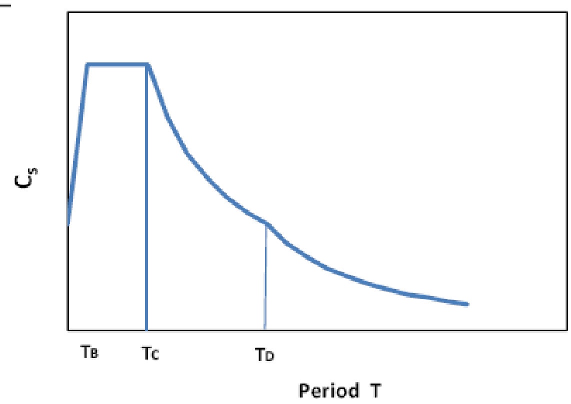
S (soil factor), T_B, T_C, T_D depends on site class

Damping correction factor, $\eta = \sqrt{10/(5+\xi)} \ge 0.55$

ξ = Damping ratio

Normalized Acceleration Response Spectrum Graph





Seismic Design Category (SDC)

Building have to be assigned a SDC based on:

- Seismic zone
- Local site condition
- Importance class

SDC – D has the most intrinsic seismic design detailing and SDC –B has the least seismic detailing requirement

	Occupancy Category I, II and III				Occupancy Category IV			
Site Class	Zone 1	Zone 2	Zone 3	Zone 4	Zone 1	Zone 2	Zone 3	Zone 4
SA	B	C	C	D	C	D	D	D
SB	B	C	D	D	C	D	D	D
SC	B	C	D	D	C	D	D	D
SD	C	D	D	D	D	D	D	D
SE, S_1, S_2	D	D	D	D	D	D	D	D

Occupancy Importance Factor (I)

Nature of Occupancy	**Occupancy Category**	**Importance Factor**
Building have low hazard to human life in the event of failure	I	1.0
Buildings except those listed in Occupancy Categories in I, III and IV	II	1.0
• Building have substantial hazard to human life in the event of failure • Buildings potential to cause a substantial economic impact or mass disruption to day to day civilian life in the event of failure • Building containing substantial quantities of toxic or explosive substances	III	1.25
Building designated as essential facilities: • Hospital, emergency shelter, power generation station • Fire, police station and emergency vehicle garage • Aviation control tower etc.	IV	1.5

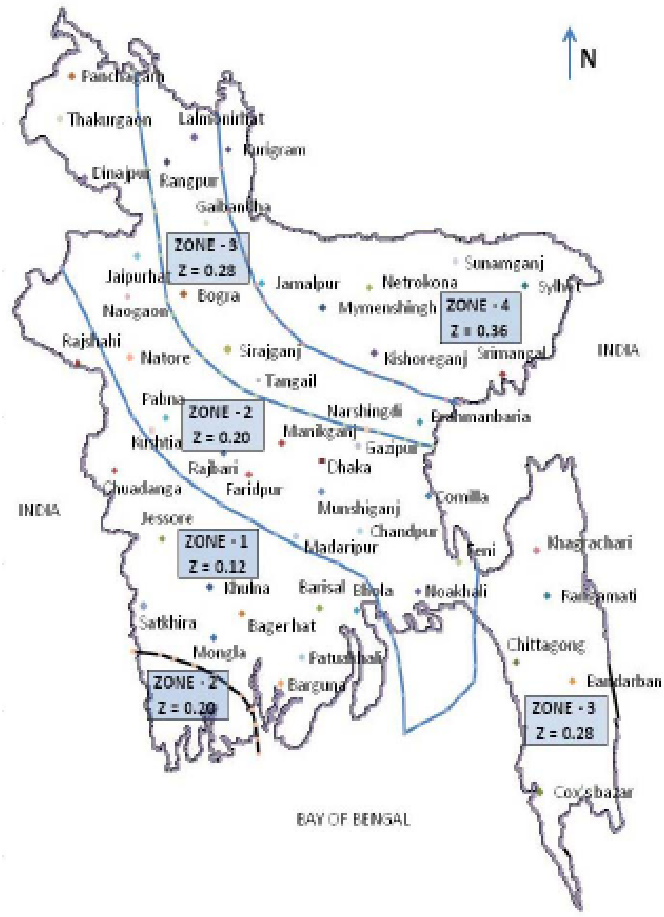
Choice of Structural System

Seismic Force Resisting System	**R**	**C_d**	**SDC-B**	**SDC-C**	**SDC-D**
			Height Limit (m)		
A. Bearing Wall System					
1. Special reinforced concrete shear wall	5	5	NL	NL	50
2. Ordinary reinforced concrete shear wall	4	4	NL	NL	NP
3. Ordinary reinforced masonry shear wall	2	1.75	NL	50	NP
4. Ordinary plain masonry shear wall	1.5	1.25	18	NP	NP
B. Building Frame System					
5. Special reinforced concrete shear wall	5	4.25	NL	NL	NP
6. Ordinary reinforced concrete shear wall	2	2	NL	50	NP
7. Ordinary reinforced masonry shear wall	1.5	1.25	18	NP	NP

Choice of Structural System

Seismic Force Resisting System	R	C_d	SDC-B	SDC-C	SDC-D
			Height Limit (m)		
C. Moment Resisting Frame System					
4. Special RC moment frame	8	5.5	NL	NL	NL
5. Intermediate RC moment frame	5	4.5	NL	NL	NP
6. Ordinary RC moment frame	3	2.5	NL	NP	NP
D. Dual Systems: SMF Capable of 25% V					
3. Special RC shear wall	7	5.5	NL	NL	NL
4. Ordinary RC shear wall	6	5	NL	NL	NP
E. Dual Systems: IMF Capable of 25% V					
3. Special RC shear wall	6.5	5	NL	NL	50
4. Ordinary RC shear wall	5.5	4.5	NL	NL	NP
F. Dual Systems: Ordinary RC Moment Frame and Ordinary RC Shear wall	4.5	4	NL	NP	NP

Zone Factor(Z)

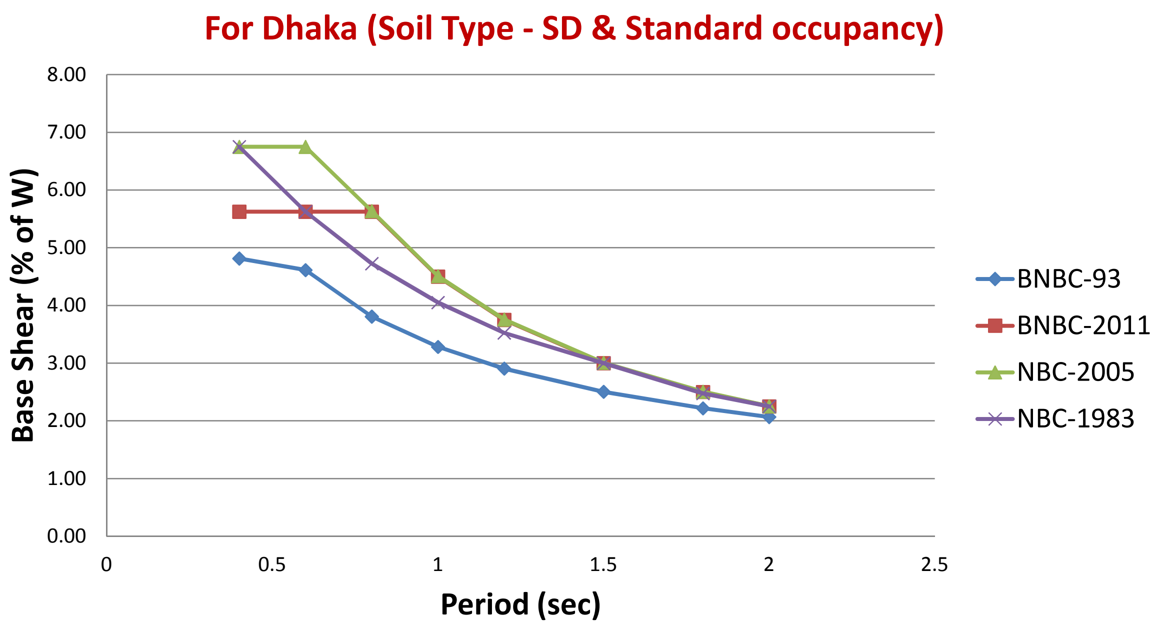


Comparison of Base Shear

For Dhaka (Soil Type - SD & Standard occupancy)

Structural period (T)	Base shear (% of wt)		% change in BNBC-2011	Base shear (% of wt) according to Indian Code	
	BNBC-93	BNBC-2011		NBC-2005	NBC-1983
0.4	4.81	5.63	16.88%	6.75	6.75
0.6	4.61	5.63	21.95%	6.75	5.63
0.8	3.81	5.63	47.73%	5.64	4.73
1	3.28	4.50	37.14%	4.51	4.05
1.2	2.91	3.75	29.06%	3.76	3.53
1.5	2.50	3.00	19.81%	3.01	3.00
1.8	2.22	2.50	12.74%	2.51	2.48
2	2.07	2.25	8.85%	2.25	2.25

Comparison of Base Shear in Various Codes

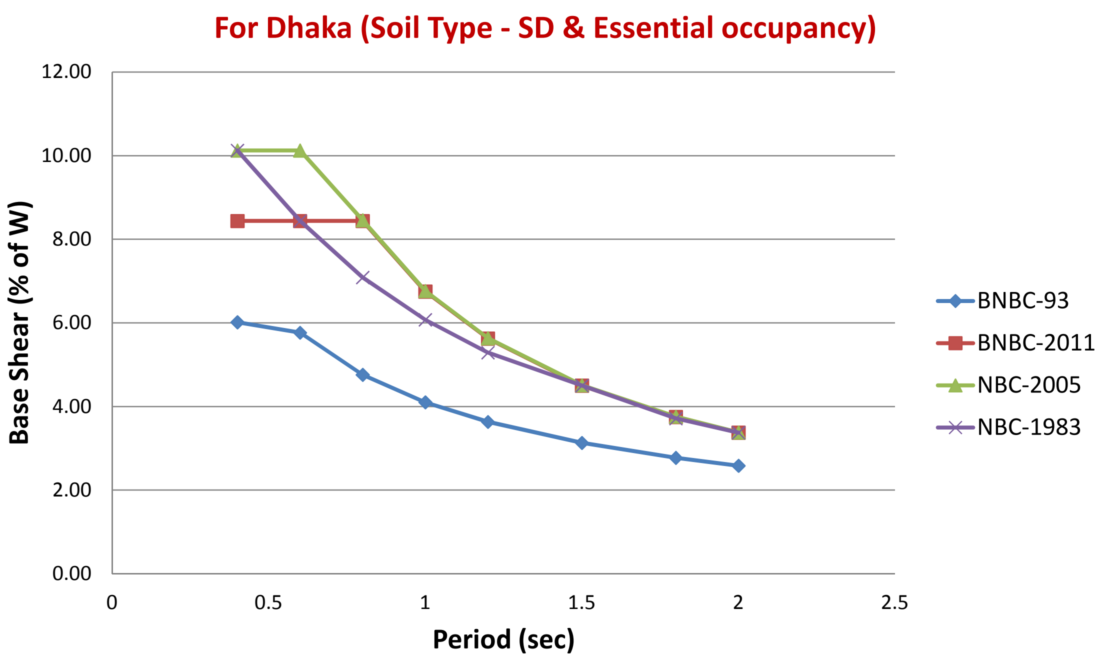


Comparison of Base Shear

For Dhaka (Soil Type - SD & Essential occupancy)

Structural period (T)	Base shear (% of wt)		% change in BNBC-2011	Base shear (% of wt) according to Indian Code	
	BNBC-93	BNBC-2011		NBC-2005	NBC-1983
0.4	6.02	8.44	40.26%	10.13	10.13
0.6	5.77	8.44	46.34%	10.13	8.44
0.8	4.76	8.44	77.28%	8.45	7.09
1	4.10	6.75	64.57%	6.76	6.08
1.2	3.63	5.63	54.87%	5.64	5.29
1.5	3.13	4.50	43.77%	4.51	4.50
1.8	2.77	3.75	35.29%	3.76	3.71
2	2.58	3.38	30.62%	3.38	3.38

Comparison of Base Shear in Various Codes



Building Period (T)

a) Structural dynamics procedure (Rayleigh method):

$$T_A = 2\pi\sqrt{\sum_{i=1}^{n} w_i\delta_i^2 \Big/ g\sum_{i=1}^{n} f_i\delta_i}$$

b) Approximate method:

$$T_B = C_t(h_n)^m$$

h_n =Height of building in meter

Structure Type	C_t	m
Concrete moment resisting frames	0.0466	0.9
Steel moment resisting frames	0.0724	0.8
Eccentrically braced steel frame	0.0731	0.75
All other structural systems	0.0488	0.75

$$T_A \le 1.4T_B$$

Vertical distribution of EQ force

$$F_x = V\frac{w_x h_x^{\,k}}{\sum_{i=1}^{n} w_i h_i^{\,k}}$$

F_x = Part of base shear force induced at level x

w_i and w_x = Seismic weight of structure at level i and x

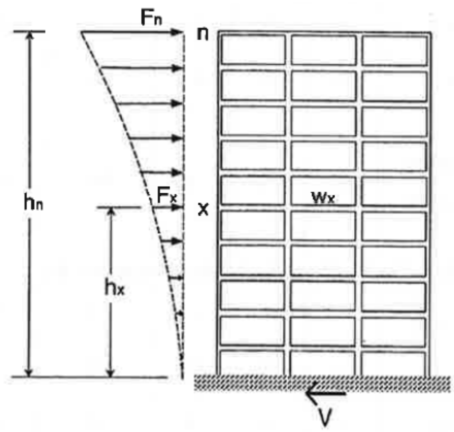
h_i and h_x = Height from base to level i and x

k = 1 for structure period ≤ 0.5 sec

= 2 for structure period ≥ 2.5 sec

= linear interpolation for other period between 1.0 and 2.0

n = number of stories



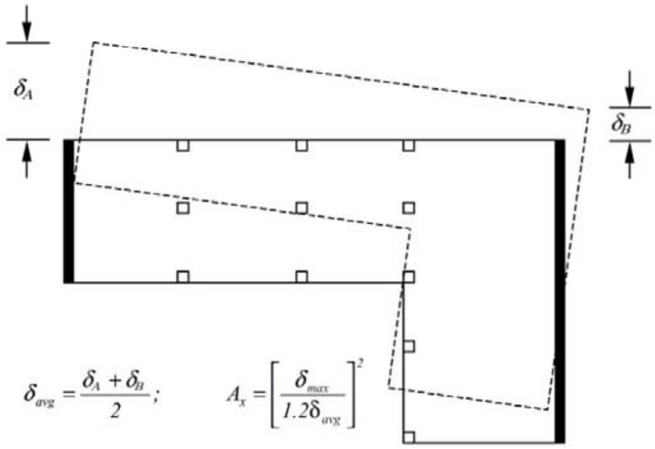
Accidental Torsional Effect

Accidental torsional moment in regular structure $M_{tai} = e_{ai} F_i$

e_{ai} = Accidental eccentricity of floor mass at level i = $\pm 0.05 L_i$

Where torsional irregularity exist in SDC-C and SDC-D increase accidental torsion, M_{ta} by A_x

$$A_x = \left[\delta_{max} / (1.2\delta_{avg})\right]^2 \leq 3.0$$



Deflection and Story Drift

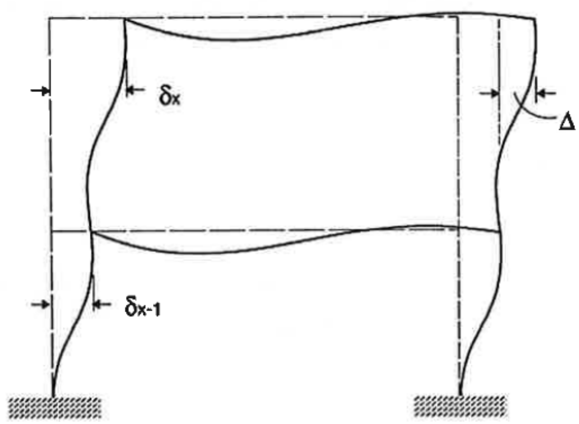
Deflection at level x, $\delta_x = \frac{C_d \delta_{xe}}{I}$

C_d = Deflection amplification factor

δ_{xe} = Deflection determined by an elastic analysis

I = Importance factor

Check deflection at center of mass



Story drift at story x, $\Delta_x = \delta_x - \delta_{x-1}$

Allowable Story Drift Limit

Structure	Occupancy Category		
	I and II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

NOTES:

1. h_{sx} is the story height below Level x.
2. There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the storey drifts.
3. Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.
4. Occupancy categories are defined in Table 1.2.1

Guideline for EQ resistant Building

1. Building shall be approximately symmetrical with respect to stiffness and mass distribution.
2. Both lateral stiffness and mass of an individual story shall remain constant or reduce gradually, without abrupt change.
3. All structural elements such as cores, structural walls or frames shall run without interruption from foundation to the top.
4. An irregular building may be subdivided into dynamically independent regular unit well separated against pounding.
5. The length by breadth ratio of the building in plan shall not be more than 4.

Effects of P-Delta

P-Delta effects are not required to be considered if stability coefficient $\theta \le 0.10$, where

$$\theta = \frac{P_X \Delta}{V_x h_{sx} C_d}$$

- P_x = Vertical load above level x (with individual load factor ≤ 1.0)
- Δ = Design story drift occurring simultaneously with V_x
- V_x = Story shear force acting between level x and x-1
- h_{sx} = Story height below level x
- C_d = Deflection amplification factor

$$\theta_{max} = \frac{0.5}{\beta C_d} \le 0.25 \qquad \text{conservatively, } \beta = 1.0$$

If $0.10 \le \theta \le \theta_{max}$ increase displacement and member forces by rational analysis or multiply by a factor $1.0/(1-\theta)$

Requirements for Static and Dynamic Analysis

Equivalent static analysis may be applied if two conditions satisfy:

1. The building period in two main horizontal direction is smaller than both $4T_c$ and 2 sec.
2. The building does not posses any vertical irregularity.

Dynamic analysis should be performed for following buildings:

1. Regular buildings with height greater than 40m in Zones – 2, 3, 4 and greater than 90m in Zone – 1.
2. Irregular buildings with height greater than 12m in zone – 2, 3, 4 and greater than 40m in Zone – 1.

Earthquake Load Combination

Following are the guidelines for combination of earthquake load in two orthogonal direction:

1. For structures of SDC-B the design seismic forces are permitted to be applied independently in each of two orthogonal direction.
2. Structures of SDC–C and D, in addition to applying requirements for SDC–B following combinations should be satisfied:
   "±100% in X-direction ±30% in Y-direction"
   "±30% in X-direction ±100% in Y-direction"

The combination which produce most unfavourable effect, shall be considered.

Vertical Earthquake Loading

Maximum vertical ground acceleration shall be taken as 50% of expected horizontal PGA.

The vertical seismic load effect E_v may be determined as :

$$E_v = 0.5(a_h)D$$

Where,

a_h = expected horizontal peak ground acceleration for design

$= (2/3)ZS$

D = effect of dead load

Load Combinations for EQ force

Common load combinations:

- 1.4D
- 1.2D + 1.6L
- 1.2D + 1.0L + 1.0E
- 0.9D + 1.0E

D = Dead load
L = Live load
E = Earthquake load

Provision for Soft Story

- Soft story problem is one of the major vertical irregularity.
- Commonly it happens in open parking floor.

Following two approaches are recommended –

1. Approach-1: Perform dynamic analysis considering strength and stiffness of infill wall and calculate inelastic deformations in members.
2. Approach -2:
   a) Carry out elastic earthquake analysis neglecting effect of infill wall
   b) Beam and column of soft story to be designed for 2.5 times shear and moment derived from elastic analysis.
   c) Symmetrically placed shear wall to be designed for 1.5 times lateral shear force calculated from elastic analysis

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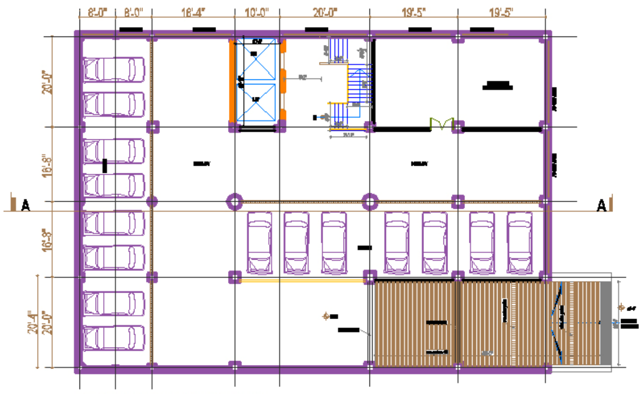
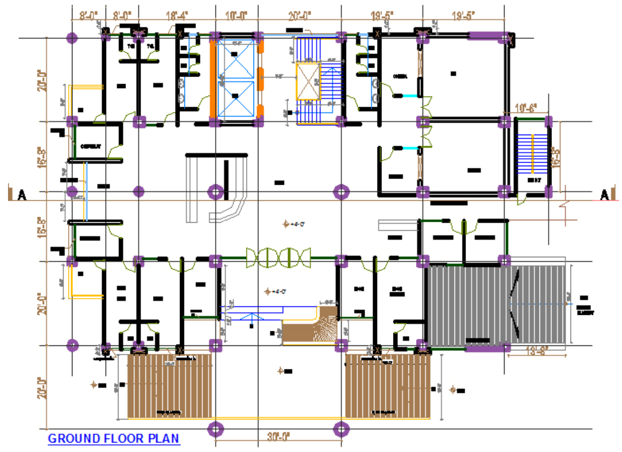
Architectural Plan view of example



SEMI BASEMENT FLOOR PLAN



GROUND FLOOR PLAN

<u>Reference Code</u>

For Analysis:	Upcoming BNBC
For Design & Detailing	ACI 318-08

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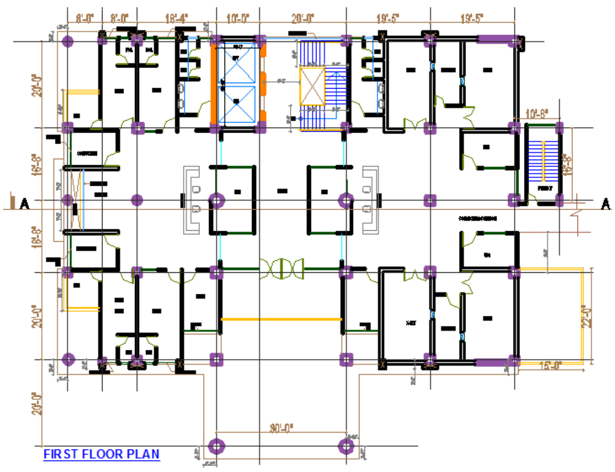
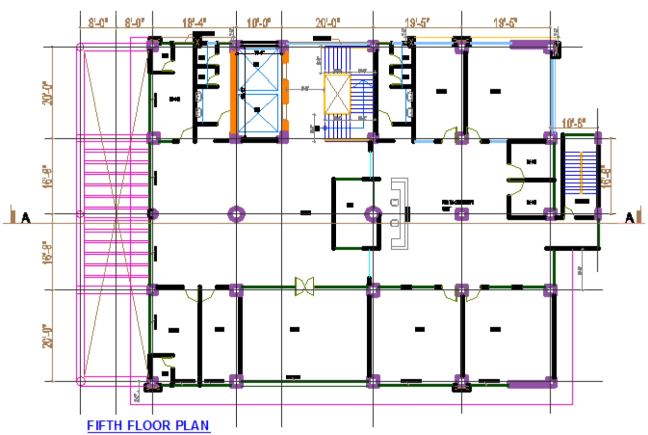
Architectural Plan view of example

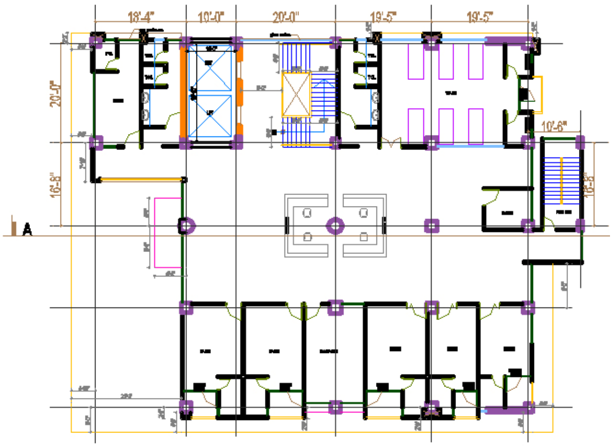


FIRST FLOOR PLAN

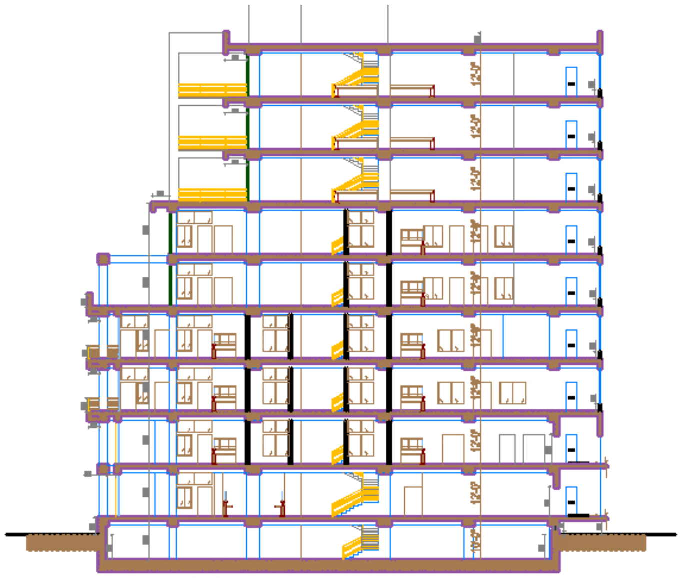


FIFTH FLOOR PLAN

Architectural Plan view of example

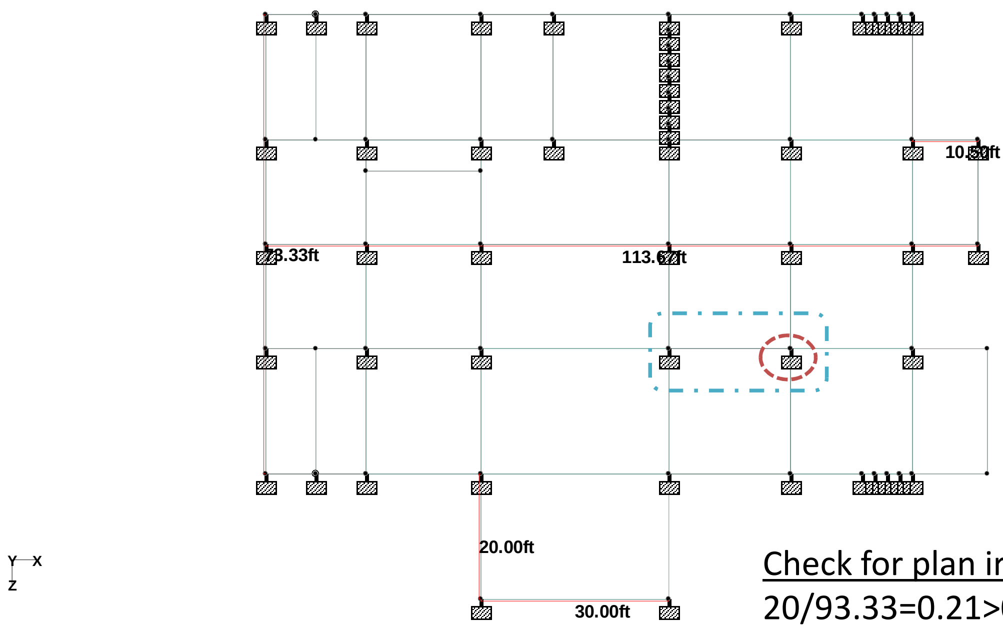


SIXTH FLOOR PLAN



SECTION A-A

Plan view of structural model



Check for plan irregularity:

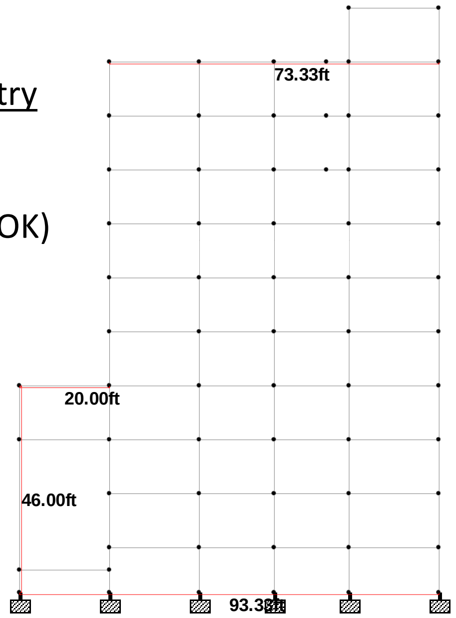
20/93.33=0.21>0.15 (Not OK)

[Ref art- 1.7.2.9(d) of BNBC 93]

Elevation of structural model

<u>Check for vertical geometry irregularity:</u>

93.33/73.33=1.27<1.30 (OK)



Selection of Seismic Design Category

1. Building located at Dhaka (Z = 0.2)
2. Hospital building (I = 1.5)
3. Soil type is SD
4. Seismic Design Category is SDC - D

Soil type	S	T_B(s)	T_C (s)	T_D (s)
SA	1.0	0.15	0.40	2.0
SB	1.2	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.4	0.15	0.50	2.0

	Occupancy Category I, II and III				Occupancy Category IV			
Site Class	Zone 1	Zone 2	Zone 3	Zone 4	Zone 1	Zone 2	Zone 3	Zone 4
SA	B	C	C	D	C	D	D	D
SB	B	C	D	D	C	D	D	D
SC	B	C	D	D	C	D	D	D
SD	C	D	D	D	D	D	D	D
SE, S_1, S_2	D	D	D	D	D	D	D	D

Selection of Structural System

Seismic Force Resisting System	R	C_d	SDC-B	SDC-C	SDC-D
			Height Limit (m)		
A. Bearing Wall System					
1. Special reinforced concrete shear wall	5	5	NL	NL	50
2. Ordinary reinforced concrete shear wall	4	4	NL	NL	NP
3. Ordinary reinforced masonry shear wall	2	1.75	NL	50	NP
4. Ordinary plain masonry shear wall	1.5	1.25	18	NP	NP
B. Building Frame System					
5. Special reinforced concrete shear wall	5	4,25	NL	NL	NP
6. Ordinary reinforced concrete shear wall	2	2	NL	50	NP
7. Ordinary reinforced masonry shear wall	1.5	1.25	18	NP	NP

Selection of Structural System

Seismic Force Resisting System	R	C_d	SDC-B	SDC-C	SDC-D
			Height Limit (m)		
C. Moment Resisting Frame System					
4. Special RC moment frame	8	5.5	NL	NL	NL
5. Intermediate RC moment frame	5	4.5	NL	NL	NP
6. Ordinary RC moment frame	3	2.5	NL	NP	NP
D. Dual Systems: SMF Capable of 25% V					
3. Special RC shear wall	7	5.5	NL	NL	NL
4. Ordinary RC shear wall	6	5	NL	NL	NP
E. Dual Systems: IMF Capable of 25% V					
2. Special RC shear wall	6.5	5	NL	NL	50
4. Ordinary RC shear wall	5.5	4.5	NL	NL	NP
F. Dual Systems: Ordinary RC Moment Frame and Ordinary RC Shear wall	4.5	4	NL	NP	NP

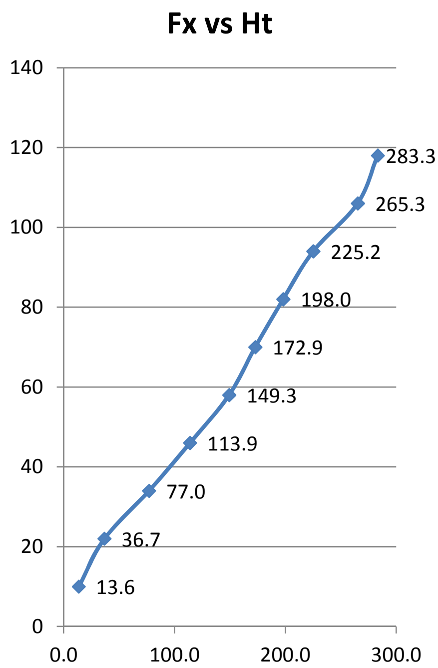
Calculation of Base Shear

5. Lateral load resisting system is SMF with special RC shear wall (R = 7)
6. S = 1.35, T_B = 0.2sec, T_C = 0.8 sec, T_D = 2.0 sec
7. Period of the structure is 1.08 sec.
8. $C_s = 2.5\eta(T_C/T) = 2.5$
9. $S_a = (2/3)(Z*I/R)C_s = 0.0714$
10. Minimum $S_a = (2/3)(Z*I)\beta = 0.04$
11. Weight, W = 20743 (DL) + 744 (25% of LL) = 21487 kip
12. Calculated Base Shear = S_aW = 1535 kip

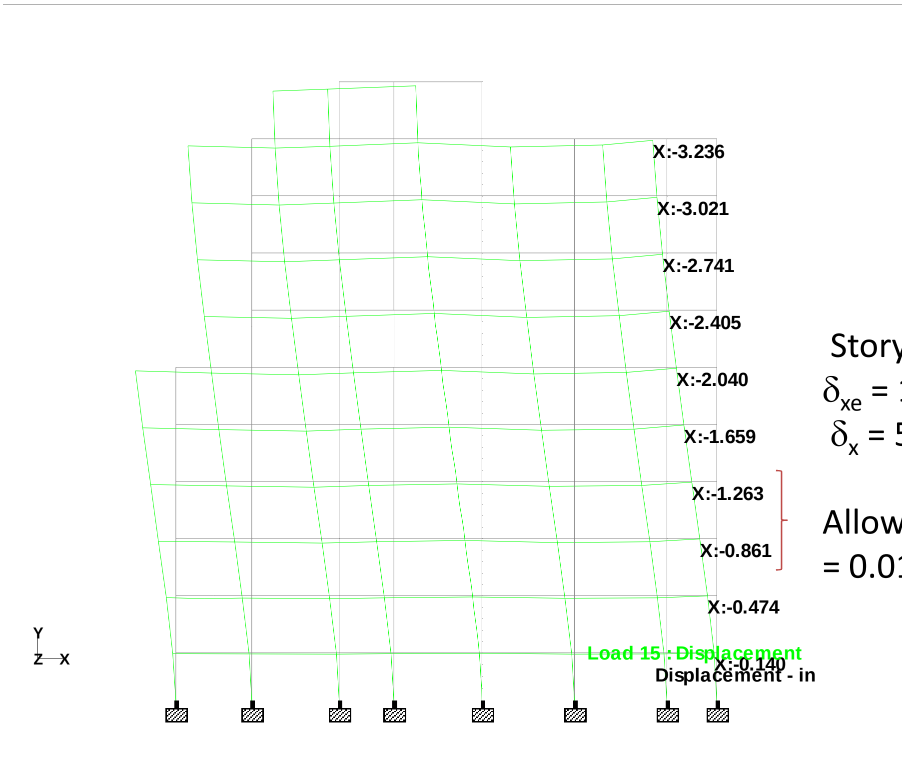
Calculation of Story Shear

K = 1.387 for T =1.08 sec

Floor level	Story weight, w_x (kip)	Height, h_x (ft)	$w_xh_x^k$	Lateral Force, F_x (kip)	Story shear, V_x (kip)
10	1677	118	1253815	283.3	283.3
9	1822	106	1173945	265.3	548.6
8	1827	94	996476	225.2	773.7
7	1942	82	876413	198.0	971.8
6	2111	70	764961	172.9	1144.6
5	2366	58	660525	149.3	1293.9
4	2491	46	504219	113.9	1407.8
3	2559	34	340590	77.0	1484.8
2	2230	22	162273	36.7	1521.4
1	2462	10	60019	13.6	1535.0
Σ	**21487**		**6793235**		



Check Storey Drift



Story drift:

δ_{xe} = 1.263 – 0.861 =0.0.402″

δ_x = 5.5*0.402/1.5 =1.474″

Allowable story drift

= 0.01h_{sx} =1.44” , Ok

Check for P-Delta Effect

P-Delta effects need not be considered if stability coefficient $\theta \leq 0.10$

$$\theta = \frac{P_X \Delta}{V_x h_{sx} C_d}$$

At ground floor level:

P_x = 20605 kip

Δ = 1.837 inch

V_x = 1521.4 kip

h_{sx} = 12ft

C_d = 5.5

So, $\theta = 0.031 < 0.1$

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$$ conservatively, $\beta = 1.0$

Here $\theta_{max} = 0.091$

If $0.10 \leq \theta \leq \theta_{max}$ increase displacement and member forces by rational analysis or multiply by a factor 1.0/(1- θ)

Principle for Design of SMF

- Design a strong-column/weak beam frame
- Avoid shear failure
- Detail for ductile behavior

21.1.4 – Concrete Properties of SMF

- 21.1.4.1 – Provisions apply to special moment frames, special structural walls, and coupling beams.
- 21.1.4.2 – Specified concrete compressive strength must be at least 3000 psi.
- 21.1.4.3 – Specified concrete compressive strength must not exceed 5000 psi for lightweight concrete.

21.1.5 – Reinforcement of SMF

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- 21.1.5.1 – Provisions apply to special moment frames, special structural walls, and coupling beams.
- 21.1.5.2 – Deformed reinforcement must satisfy ASTM A706. Grades 40 and 60 of ASTM A 615 are permitted if:
  - The actual yield stress does not exceed the nominal yield stress by more than 18 ksi.
  - The ratio of the actual tensile strength to actual yield stress exceeds 1.25.

21.5 – Beams in Special Moment Frames

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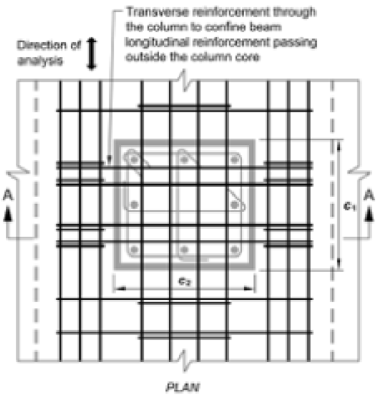
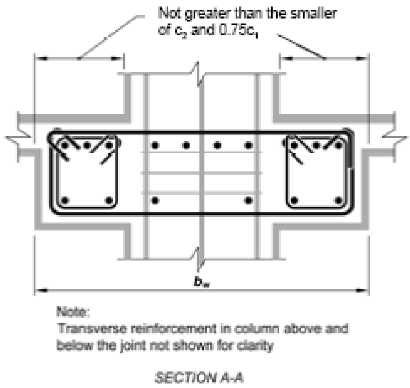
- A beam is defined as any frame member that resists earthquake-induced forces and is proportioned primarily to resist flexure.
- Beams must satisfy the following:
  - Factored axial compressive force must not exceed $A_g f_c'/10$.
  - Clear span must be more than 4 times the effective depth.
  - Width of member must not be less than the smaller of 0.3h and 10 in.

21.5 – Beams in Special Moment Frames

- 21.5.1.4 relaxed to permit wide beams.
  - $b_{w,max} = min\ (3c_2,\ c_2{+}1.5c_1)$

- 21.7.3.3 added to address confinement of longitudinal beam reinforcement located beyond column core.





21.5.2 – Longitudinal Reinforcement

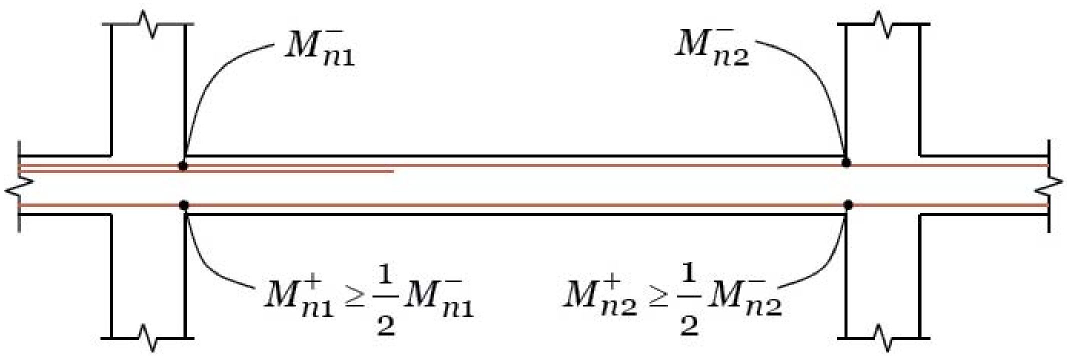
All locations:

$$A_s \geq \frac{3\sqrt{f'_c}}{f_y} b_w d \geq \frac{200}{f_y} b_w d$$

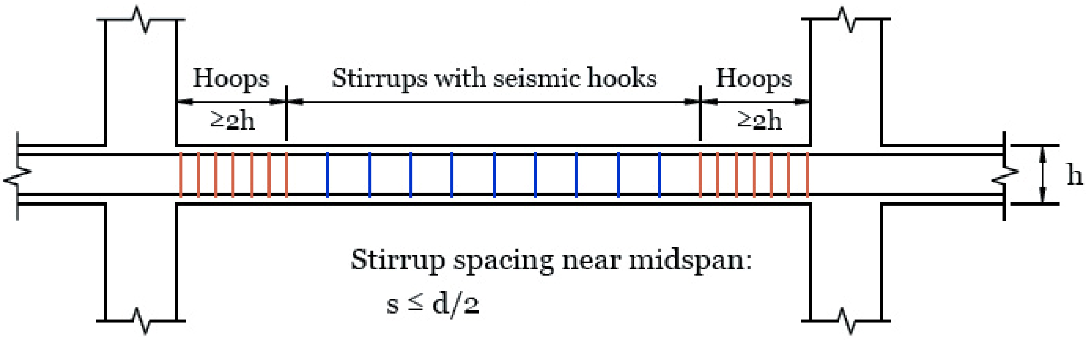
Minimum of two continuous bars per face

$$M_n \geq \frac{1}{4} \max\left(M_{n1}^-, M_{n2}^-\right)$$

$$\rho \leq 0.025$$



21.5.3 – Transverse Reinforceme...



Hoop spacing near joint:

$s \le \min(d/4, 8d_b, 24d_b, 12\text{ in.})$ = 5", 6", 9", 12" for beam size 15"x24"

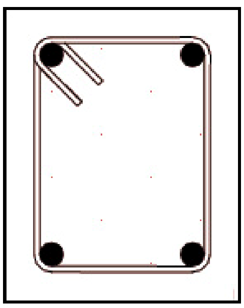
Hoop bar

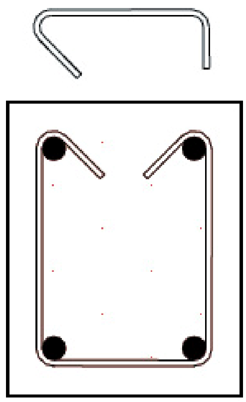
Smallest longitudinal bar

First hoop is located no more than 2 in. from the face of the support

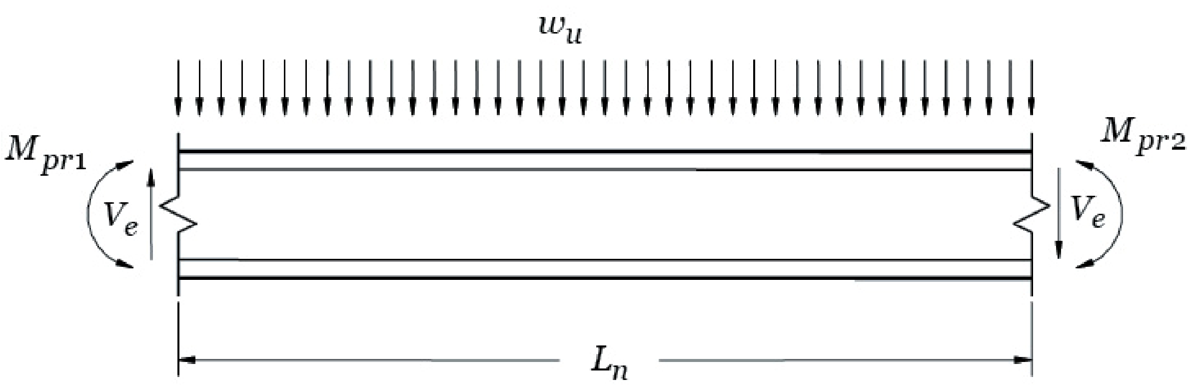
21.5.3 – Transverse Reinforceme...

- Hoops in beams are permitted to be made of two pieces of reinforcement: a stirrups having seismic hooks at both ends and a cross tie.





21.5.4 – Shear Strength Requirements



Capacity Design Approach:

$$V_e = \frac{M_{pr1} + M_{pr2}}{L_n} \pm \frac{w_u L_n}{2}$$

21.5.4 – Shear Strength Requirements

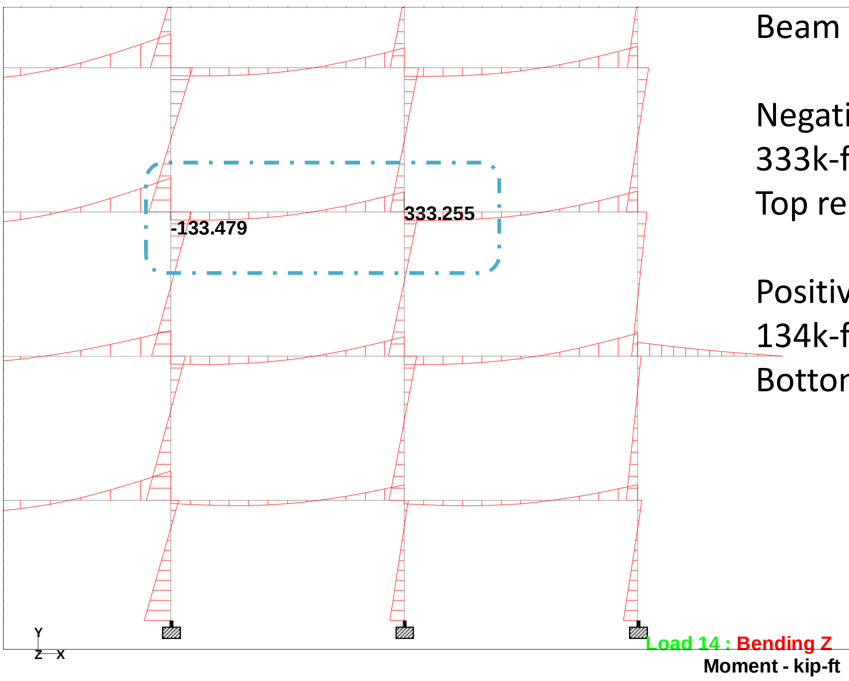
V_e = design shear force (factored shear)

M_{pr} = probable flexural strength, calculated using a stress in the reinforcement of 1.25 f_y and a strength reduction factor of 1.0.

21.5.4 – Shear Strength Requirements

- Transverse reinforcement in the regions where hoops are required shall be proportioned to resist shear assuming that
  $V_c = 0$ when both of the following conditions occur:
  - The earthquake-induced shear force represents at least 50% of the required shear strength.
  - The factored axial compressive force including earthquake effects is less than $A_g f_c'/20$.

Beam Bending moment

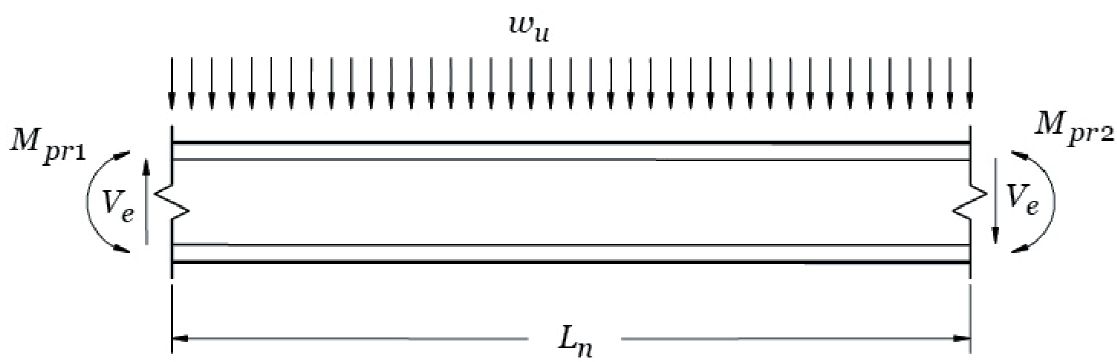


Beam size = 15” x 24”

Negative moment of beam = 333k-ft
Top rebar = 6-d25

Positive moment of beam = 134k-ft
Bottom rebar = 2-d25+2-d20

Earthquake Induced Shear Force



$M_{pr} = A_s * 1.25F_y * (d – a/2)$

M_{pr1} = 468 k-ft and M_{pr2} = 286 k-ft

$V_{eq} = (M_{pr1} + M_{pr2})/L$ = 39.0 kip

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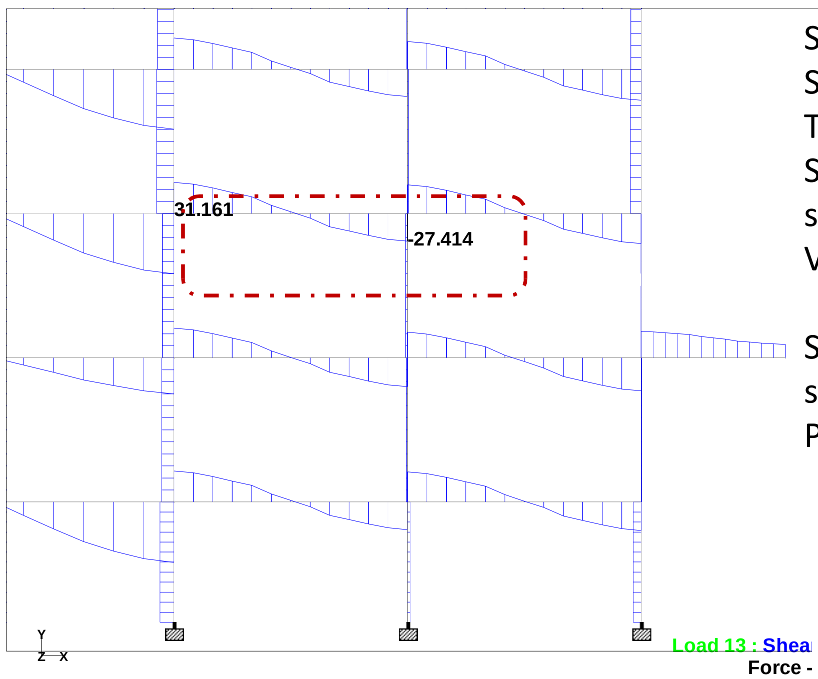
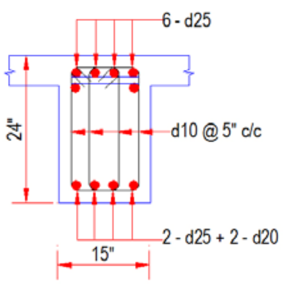
Beam Shear Force



Shear for service load = 31.2k
Shear for EQ = 39.0k
Total shear force = 70.2k
Since 39.0>31.2 and P = 8.7 k
so Vc = 0
Vs = 70.2k

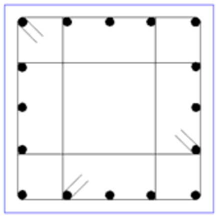
So, required hoop spacing
s = 3.37” (2leg-d10)
Provide 4-leg d10 @ 5” c/c



21.6 – Column in Special Moment Frames

- A column is defined as any frame member that resists earthquake-induced forces and has a factored axial force in any load combination that exceeds $A_g f_c'/10$.
- Columns must satisfy the following:
  - Shorter cross-sectional dimension must be at least 12 in.
  - Aspect ratio for the column must not be less than 0.4.

For axial load 1030 kip
and Moment 258k-ft
Column designed as
Size = 24”x24”
Main rebar = 16-d25
Hoop = d10 @ 4” c/c



21.6.2.2 – Strong Columns/Weak Beams

- A strong column-weak beam system must satisfy:

$$\Sigma M_{nc} \geq (6/5)\Sigma M_{nb}$$

M_{nc} = sum of moments at the faces of the joint corresponding to the nominal flexural strength of the columns framing into that joint.

M_{nb} = sum of moments at the faces of the joint corresponding to the nominal flexural strength of the girders framing into that joint. In T-beam construction, where the slab is in tension under the moments at the face of the joint, slab reinforcement within the effective slab width defined in 8.10 shall be assumed to contribute to the flexural strength if the slab reinforcement is developed at the critical section for flexure.

21.6.2.2 – Strong Columns/Weak Beams

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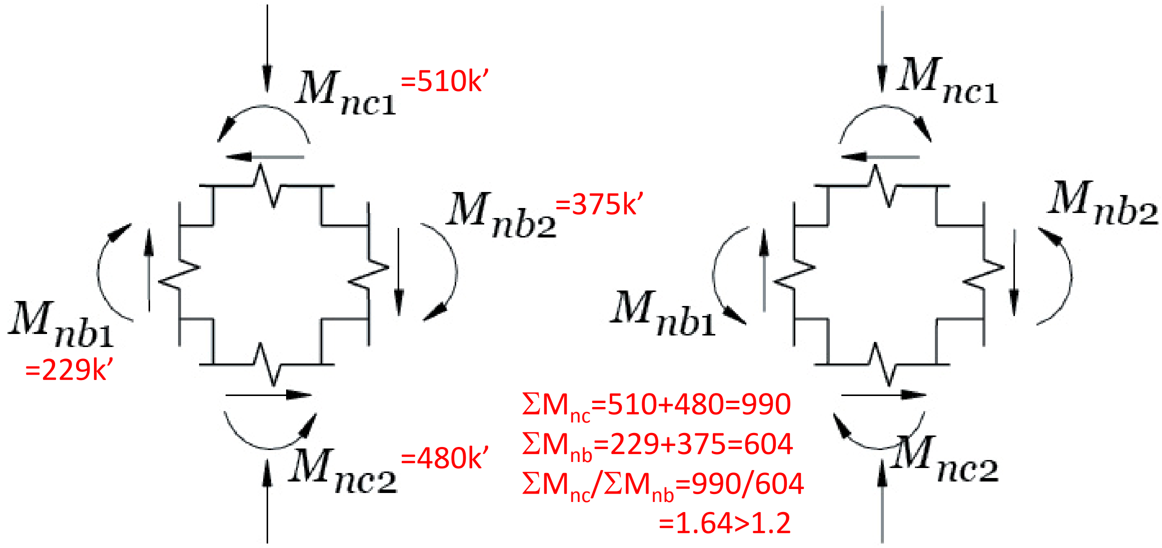

The nominal flexural capacities of the members are summed such that column moments oppose the beam moments. The column strengths must satisfy the relationship for beam moments acting in both directions.

21.6.2.2 – Strong Columns/Weak Beams

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- If the columns do not satisfy the requirements for strong columns, the columns must satisfy the provisions in 21.13.
- In addition, the lateral strength and stiffness of columns that do not satisfy 21.6.2.2 must be ignored when calculating the strength and stiffness of the structure.

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21.6.3 – Longitudinal Reinforcement

- The longitudinal reinforcement ratio must not be less than 0.01 nor more than 0.06.
- Lap splices are only permitted within the center half of the member and must be proportioned as tension splices.

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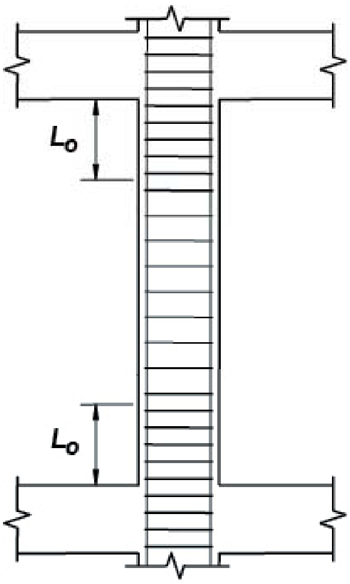
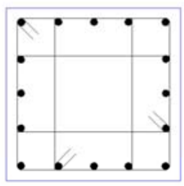
21.6.4.1 – Transverse Reinforcement

L_o is the largest of:

h, ***b***, $\frac{L_u}{6}$, 18 in.

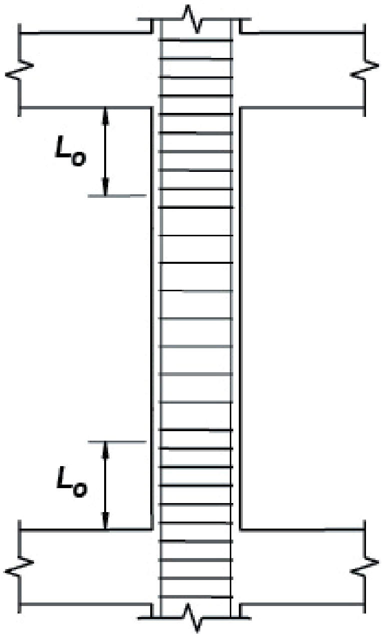
=24”, 24”, 20.5”, 18”
for column size is 24”x 24”
And floor height 12’-0”





24" X 24"
16 - d25
d10 @ 4" c/c

21.6.4.3 – Spacing of Transverse Reinforcement



Within L_o, s must not exceed the smallest of:

$$\frac{b}{4},\ \frac{h}{4},\ 6d_b,\ s_o$$ =6", 6", 6", s (req)

For column size is 24"x 24"

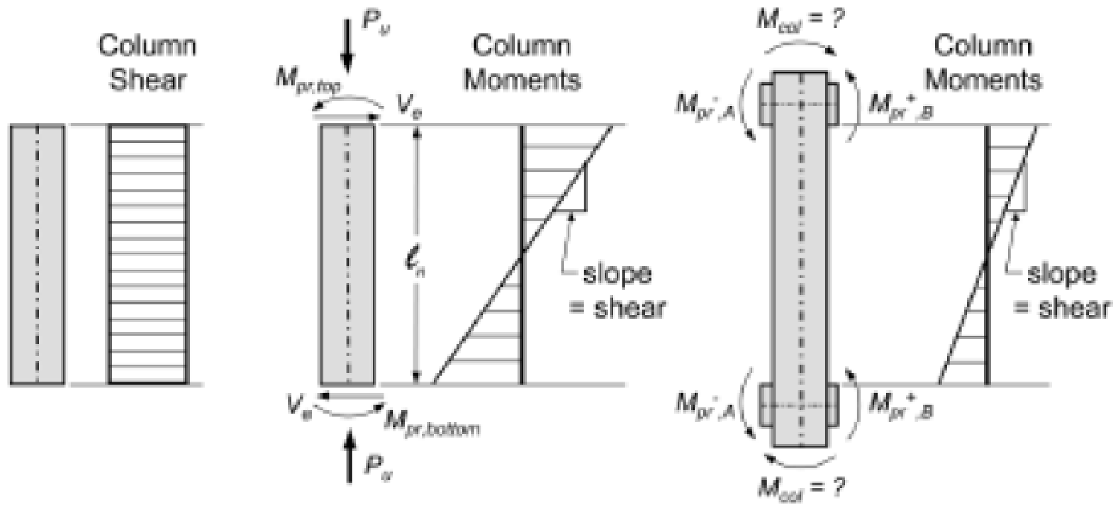
d_b = diameter of smallest longitudinal bar.

$$s_o = 4 + \left(\frac{14 - h_x}{3}\right)$$ =4.17

$$s_o \leq 6 \text{ in.}$$

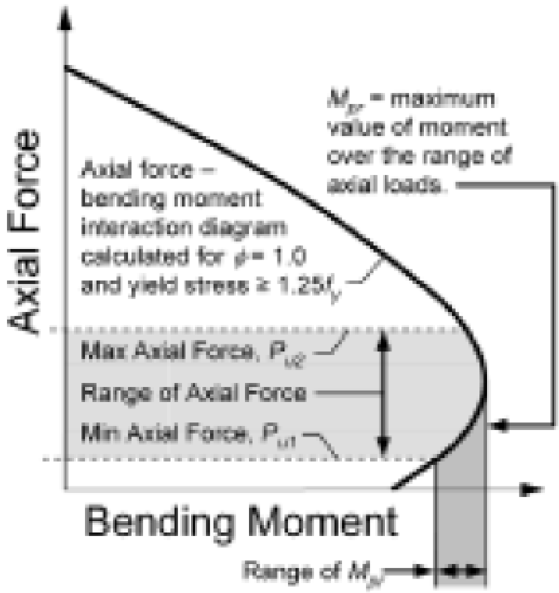
s_o need not be taken less than 4 in.

21.6.5 – Calculation of Column Shear



(a) From analysis (b) Column hinging (c) Beam hinging

Probable Moment (M_{pr}) in Columns



21.6.5.2 – Shear Strength Requirements

- Transverse reinforcement over the length L_o, shall be proportioned to resist shear assuming that V_c = 0 when both of the following conditions occur:
  - The earthquake-induced shear force represents at least 50% of the required shear strength.
  - The factored axial compressive force, P_u, including earthquake effects is less than $A_g f_c'/20$.

21.6.6.4(b) – Rectangular Hoops

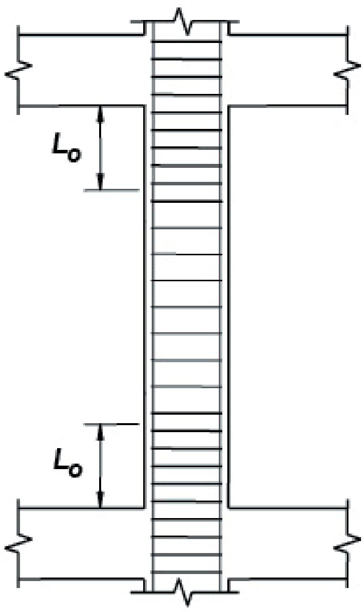
- The total cross-sectional area of rectangular hoop reinforcement must not be less than the larger of:

$$A_{sh} = 0.3\left(sb_c \frac{f'_c}{f_{yt}}\right)\left(\frac{A_g}{A_{ch}} - 1\right)$$ =0.465 < 0.48

$$A_{sh} = 0.09\left(sb_c \frac{f'_c}{f_{yt}}\right)$$ =0.456

- A_{ch} = cross-sectional area of a structural member measured to the outside edges of transverse reinforcement
- b_c = cross-sectional dimension of column core measured to the outside edges of transverse reinforcement

21.6.4.5 – Transverse Reinforcement



Outside L_o, the column shall contain spiral or hoop reinforcement satisfying 7.10, unless a larger amount of transverse reinforcement is required by 21.6.3.2 or 21.6.5.
s shall not exceed the smaller of:

$6d_b$, 6 in.

d_b = diameter of smallest longitudinal bar.

Typical Column Transverse Reinforcement Requirement

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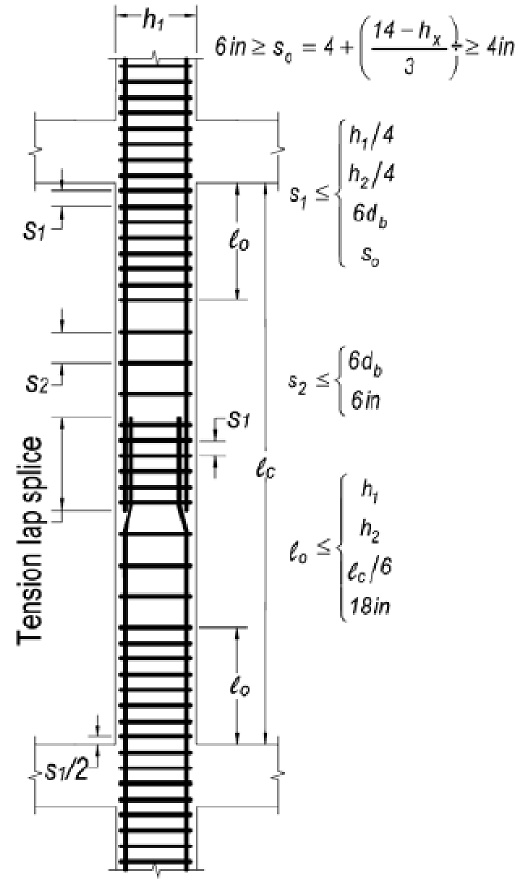

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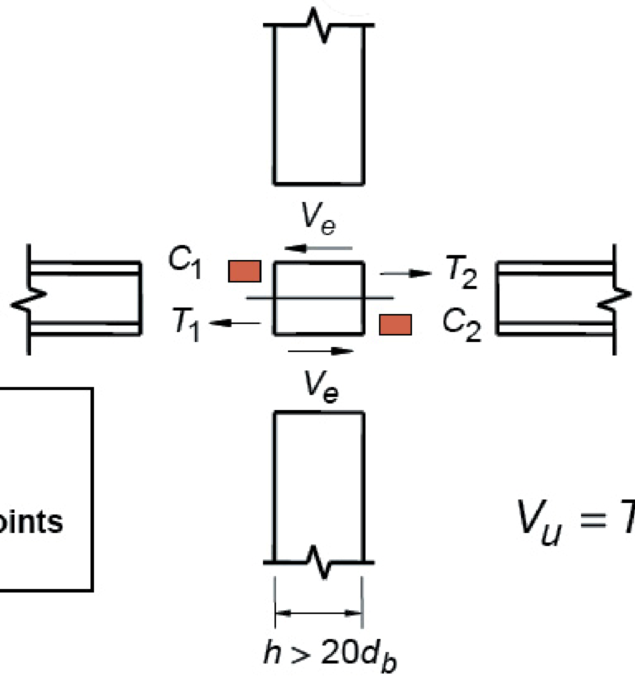
21.7 – Beam-Column Joints



$\phi = 0.85$

for shear in joints

$$V_u = T_1 + T_2 - V_e$$

21.7.2 – General Requirements

- Forces in longitudinal beam reinforcement at the joint face shall be determined by assuming that the stress in the flexural reinforcement is 1.25 fy.
- Beam reinforcement that terminates in a beam-column joint must extend to the far face of the confined core and be anchored in tension per 21.7.5 or in compression per Chapter 12.
- Where longitudinal beam reinforcement extends through a beam-column joint, the column dimensions parallel to the beam reinforcement shall not exceed 20 times the diameter of the largest longitudinal beam.

21.7.3.1~2 – Transverse Reinforcement

- The closely-spaced transverse reinforcement required near the ends of a column must be continued through the joint., except as permitted in 21.7.3.2.

- Where beams frame into all four sides of a joint and where the width of each beam is at least 75% of the column width, the amount of transverse reinforcement may be reduced by 50% and the spacing may be increased to 6 in. within the overall depth of the shallowest beam.

21.7.3.3 – Transverse Reinforcement

- Longitudinal beam reinforcement outside the column core must also be confined by transverse reinforcement that passes through the column.

- This transverse reinforcement must satisfy the spacing required by 21.5.3.2. and the requirements of 21.5.3.3 and 21.5.3.6.

21.7.4 – Shear Strength of Joint

The nominal shear strength of the joint shall not exceed the values given below:

• Joints confined on all four faces	$20\sqrt{f'_c}A_j$
• Joints confined on three faces or on two opposite faces	$15\sqrt{f'_c}A_j$
• Other joints	$12\sqrt{f'_c}A_j$

It is not possible to increase the shear strength of the joint by adding more reinforcement.

21.7.4 – Shear Strength of Joint

- A beam that frames into the face of a joint is considered to provide confinement to the joint if the area of the beam covers at least 75% of the face of the joint.

- Extensions of beams at least h beyond the joint face are considered to provide confinement. Extensions of beams must satisfy 21.5.1.3, 21.5.2.1, 21.5.3.2, 21.5.3.3, and 21.5.3.6.

21.7.4 – Shear Strength of Joint

The area of the joint, *Aj, is calculated as the joint* depth times the effective joint width.

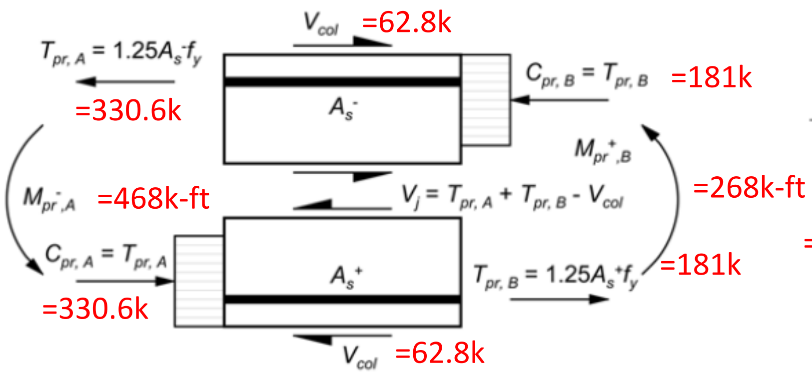
- Joint depth is the overall depth of the column, *h.*

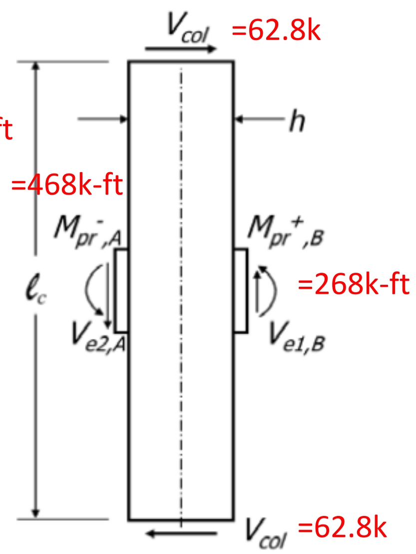
- Effective joint width is the overall width of the column, *b, except where a beam frames into a wider column.*

The effective joint width shall not exceed the smaller of the followings:

(a) Beam width plus joint depth.
(b) Twice the smaller perpendicular direction from the longitudinal
axis of the beam to the side of the column.

Calculation of joint strength





$$V_{col} = [(M_{pr}^+{}_{,B} + M_{pr}^-{}_{,A}) + (V_{e2,A} + V_{e1,B})\frac{h}{2}]/\ell_c$$

$M^-_{pr,A}$=1.25×4.56×58×(20-3)/12
=468k-ft

$T_{pr,A}$=1.25×4.56×58=330.6k

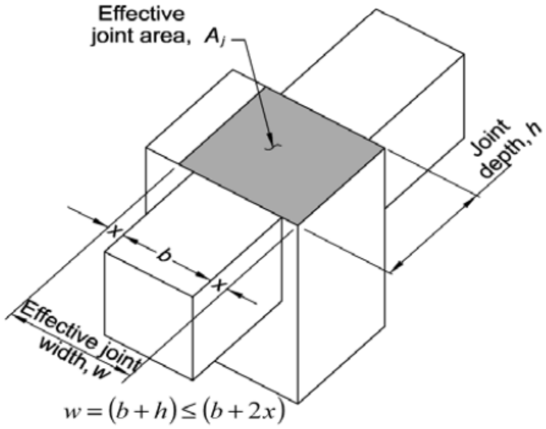
V_{col}=(468+286)/12=62.8k

Vj=330.6 + 181 – 62.8 = 448.8k

Calculation of joint strength

If beam passes through centre of the column, then
b =24" and h=24"
So, A_j = 24x24=576
φVc=12√f'c Aj
=348k < 448.8k, Not OK

If beam passes through either edges of the column, then
b =15" and h=24"
So, A_j = 15x24=360
φVc=12√f'c Aj
=216.8k < 448.8k, Not OK



21.7.5.1 – Development length of Hooked bar

- The development length for a bar with a 90° hook shall not be less than the largest of:
  - 8 $\boldsymbol{d_b}$
  - 6 in.
  - $\dfrac{f_y d_b}{65\sqrt{f'_c}}$

#3 through #11 bars
Normal weight concrete

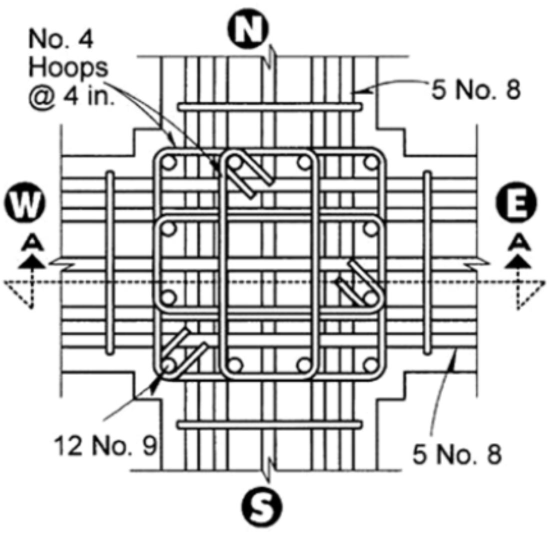
If f_y=400MPa & f'_c=3500
For 25mmϕ bar l_{dh}=15" &
For 20mmϕ bar l_{dh}=12"

- The 90° hook must be located within the confined core of a column or boundary element.

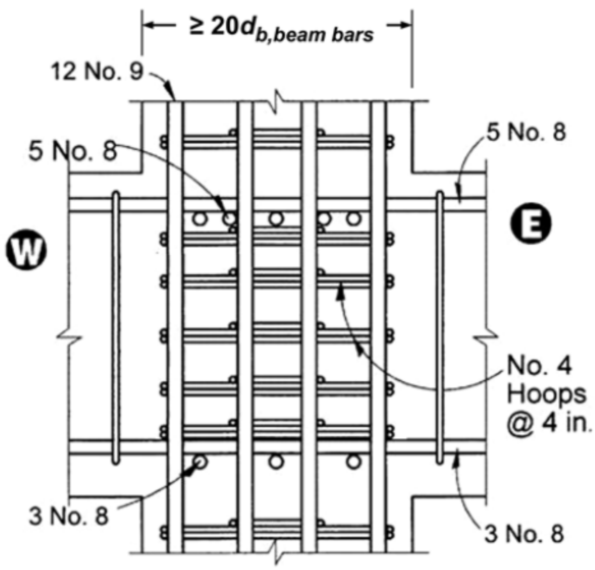
21.7.5.2 – Development length of Straight bar

- The development length of a straight bar in tension (#3 through #11) must not be less than the larger of (a) and (b):
  - (a) 2.5 times the development length for a hooked bar if the depth of concrete does not exceed 12 in.,
  - (b) 3.5 times the development length for a hooked bar if the depth of concrete exceeds 12 in.
- Straight bars terminated in a joint must pass through the confined core of a column or boundary element.
- Any portion of the straight embedded length that is not within the confined core must be increased by a factor of 1.6.

Typical Interior Joint Detailing

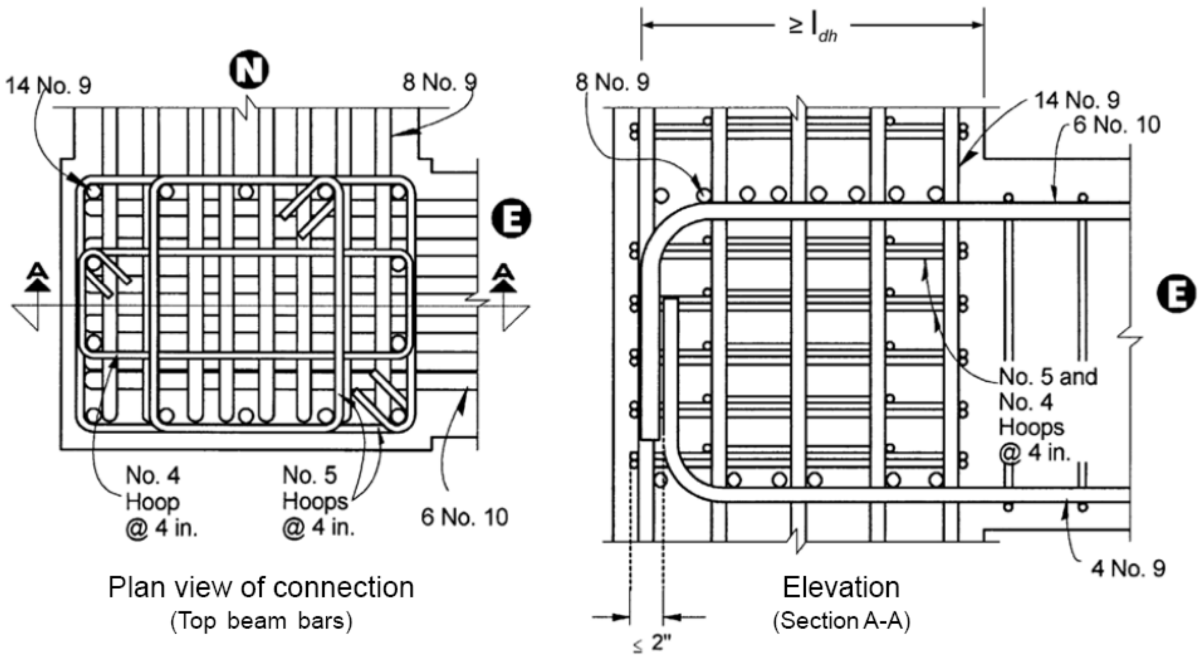


Plan view of connection
(Top beam bars)



Elevation
(Section A-A)

Typical Exterior Joint Detailing



Flow Diagram for SMF (Brief)

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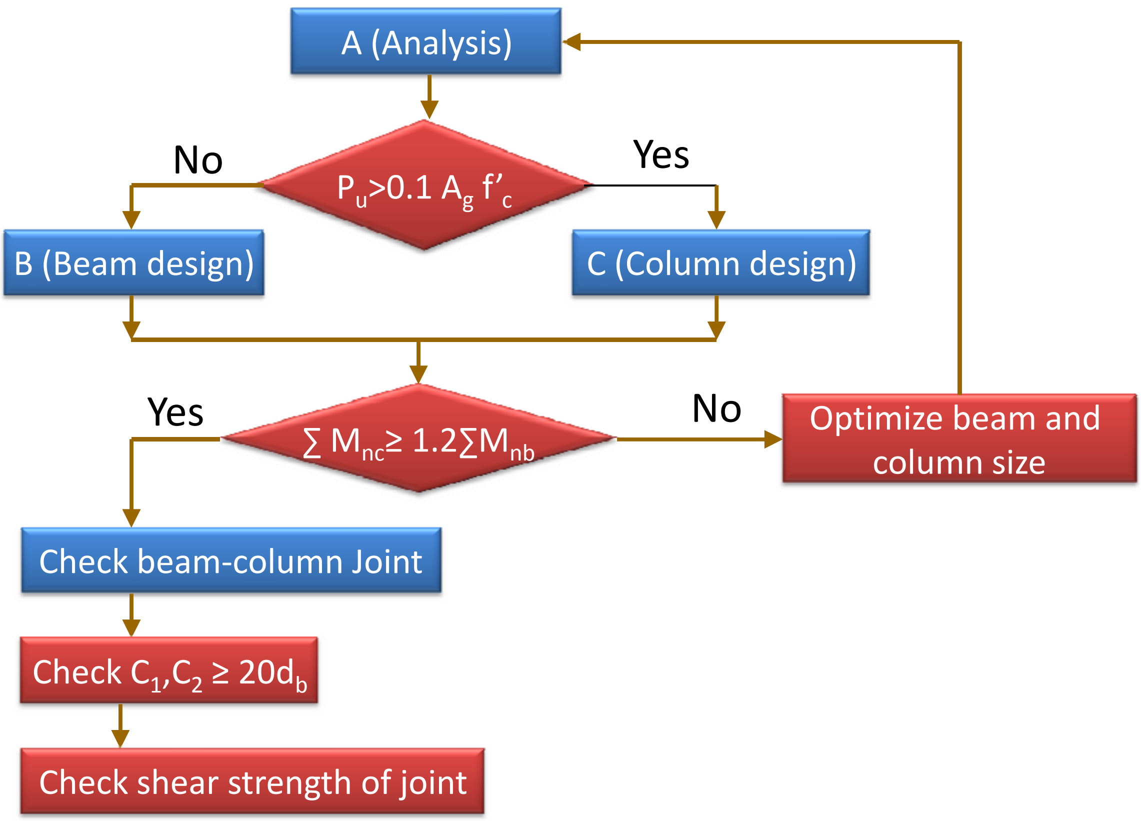

Flow Diagram for SMF (A)

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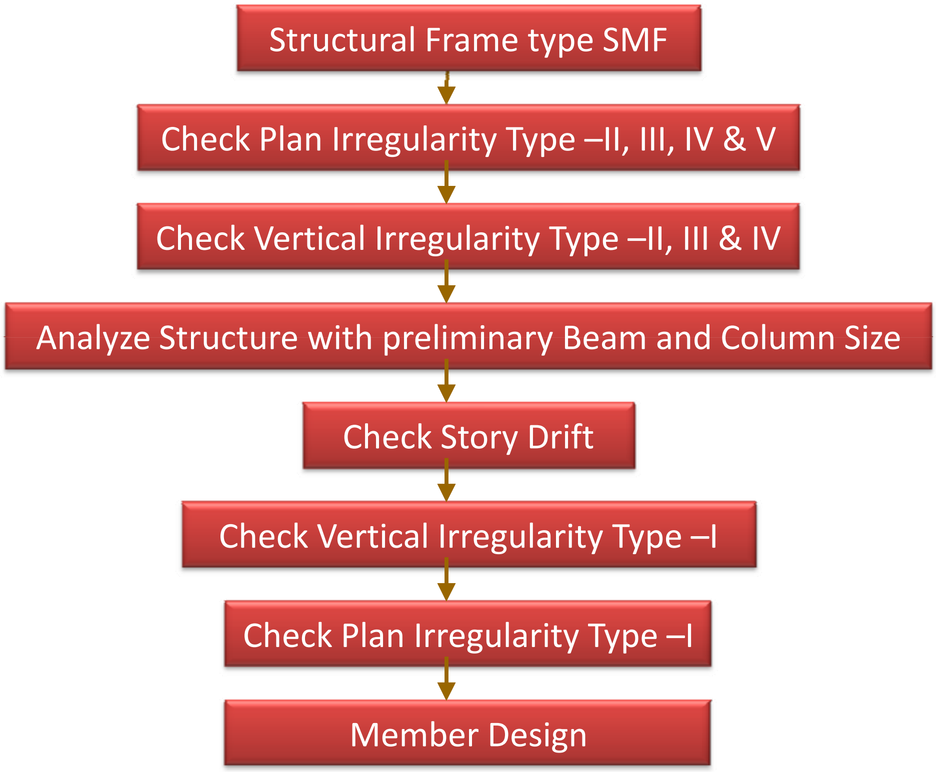

Flow Diagram for SMF (B)

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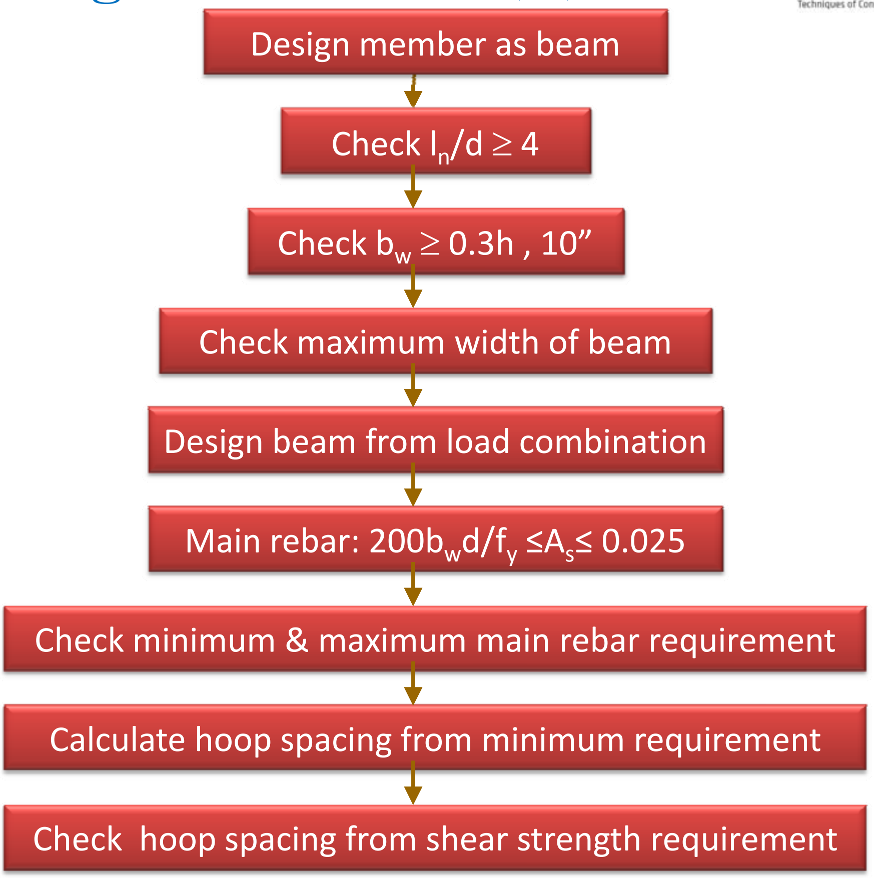

Flow Diagram for SMF (C)

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Design member as column

↓

Check $C_1/C_2 \geq 0.4$

↓

Check $C_1, C_2 \geq 12''$

↓

Design column from load combination

↓

Main rebar: $0.01\,A_g \leq A_s \leq 0.06\,A_g$

↓

Calculate hoop spacing from minimum requirement

↓

Calculate hoop/spiral spacing from volumetric ratio

↓

Calculate hoop/spiral spacing from shear strength requirement

↓

Check vertical irrelarity type - V

Thank you

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(CAPACITY DEVELOPMENT ON NATURAL DISASTER RESISTANT TECHNIQUES OF CONSTRUCTION AND RETROFITTING FOR PUBLIC BUILDINGS)

TECHNICAL CO-OPERATION PROJECT BETWEEN –PWD & JICA

PRESENTATION ON

EFFECTS OF INFILL BRICK MASONRY IN RC FRAME

PRESENTED BY
MD. **JAHID**UL ISLAM KHAN
MEMBER, WT-2, CNCRP

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CONTENTS

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INTRODUCTION

3

- Masonry infill walls are provided within the reinforced concrete structures without being analyzed as a combination of concrete and brick elements, though in reality they act as a single unit during earthquakes.
- The performance of such structures during earthquakes have proved to be superior in comparison to the bare frames in terms of stiffness, strength and energy dissipation.

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INTRODUCTION (Cont…)

4

- There are plenty of researches done so far for infilled frames, however partially infilled frames are still the topic of interest.
- Though it has been understood that the infills play significant role in enhancing the lateral stiffness of complete structure, the past experience in various earthquakes have proved that the partially infilled framed structures somehow are affected adversely.

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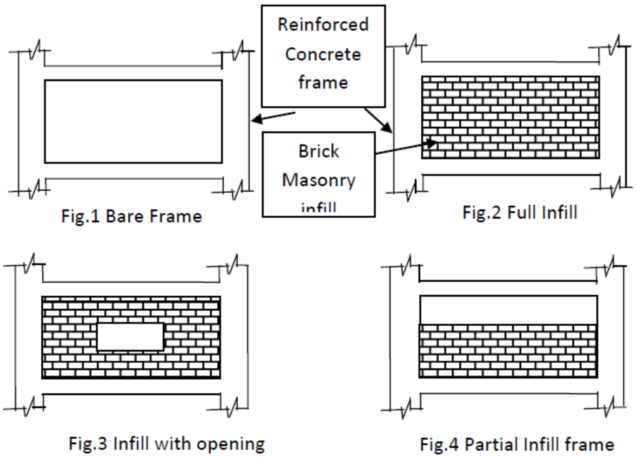
CONVENTIONAL PRACTICES

5



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PROPERTIES OF INFILL WALLS

6

- If the infill walls are very light and flexible, or completely isolated from the RC frame, presence of infill does not affect the structural response of the system.
- Infill walls are expected to remain in the elastic range.
- Infill walls are expected to suffer significant damage during the seismic event.

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ADVANTAGES OF INFILL WALLS

7

- Provides durable and economical partitions.
- Higher stiffness and lower displacement.
- Higher strength.
- Lower ductility requirements for frame.
- Frame design for small lateral loads.
- Reduce contribution of frame in lateral resisting.

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DESIGN PRACTICES OF INFILL WALLS

8

- Infills are adequately separated from the RC frame such that they do not interfere with the frame under lateral deformations.
- Infills are built integral with the RC frame, but considered as non-structural elements.
- Infills are built integral with the RC frame, and considered as structural elements.

DIFFICULTIES IN DESIGN

9

- Computational complexity.
- Structural uncertainties.
- The non-linear behaviour of infilled frames.
- Various cracking patterns and concentration of forces in structural components.

EFFECTS OF INFILL WALLS

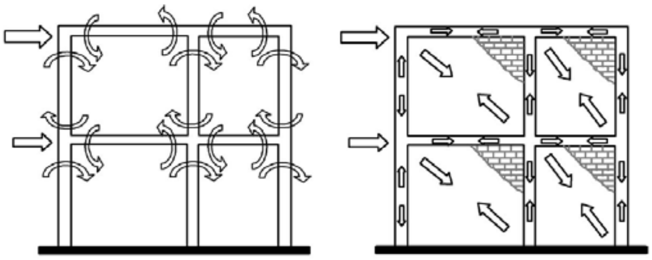
10

- Unequal distribution of lateral forces.
- Vertical irregularities in strength and stiffness.
- Horizontal irregularities.
- Inducing the effect of short column or captive column in infilled frame.

BEHAVIOUR OF INFILL WALLS

11

- The structural load transfer mechanism is changed from frame action to predominant truss action.



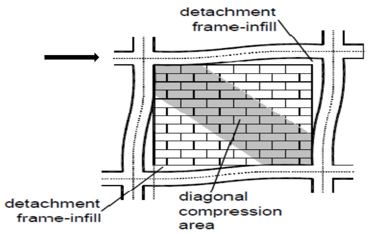
PREDOMINANT FRAME ACTION

PREDOMINANT TRUSS ACTION

BEHAVIOUR OF INFILL WALLS (Cont…)

12

- The state of stress in the infill gives rise to a principal compressive stress along the diagonal and a principal tensile stress in the perpendicular direction.



- When infills are strong, strength contributed by the infills may be comparable to the strength of the bare frame itself.

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BEHAVIOUR OF INFILL WALLS (Cont…)

13

IN-PLANE BEHAVIOUR

- The in-plane capacity of the wall depend on the relative strength of the masonry and the mortar.
- The level of the axial load significantly controls the type of failure.
- The crack propagation either follows the mortar joints or passes through the masonry units, or both.

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BEHAVIOUR OF INFILL WALLS (Cont…)

14

OUT-OF-PLANE BEHAVIOUR

- Crushing along the edges for low height to thickness ratio.
- Snap-through (small effect of arching) for high height to thickness ratio i.e. approximately between 20 and 30.

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BEHAVIOUR OF INFILL WALLS (Cont…)

15

PARTIALLY INFILLED FRAME

- In majority of hospitals, academic institutions and commercial complexes, partial infills are provided to attain light within the rooms.
- It is observed that such walls on one hand contribute in enhancing the lateral stiffness of the structure while on the other hand they play ironic role with an adverse effect called "short column effect".

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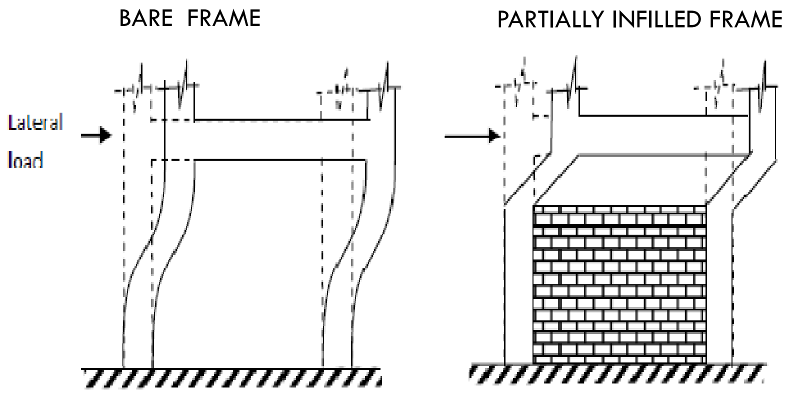
BEHAVIOUR OF INFILL WALLS (Cont…)

16



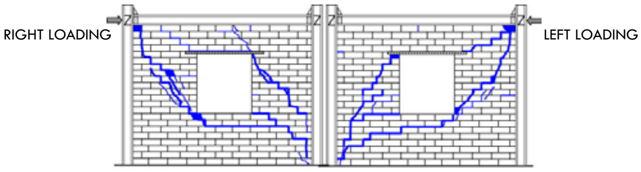
LATERAL DEFORMATION PATTERN

BEHAVIOUR OF INFILL WALLS (Cont…)

17

FRAMES WITH OPENNING

- In most cases, door or window openings are provided in masonry infill panels because of the functional and ventilation requirements of buildings.
- Introducing openings in an infill wall alters its behavior and adds complexity in behavior.

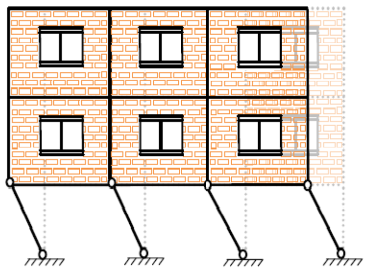


BEHAVIOUR OF INFILL WALLS (Cont…)

18

SOFT STOREY

One of the main reasons of structural failure due to earthquakes is discontinuity of lateral force resisting elements like bracing, shear wall or infill in the first storey.



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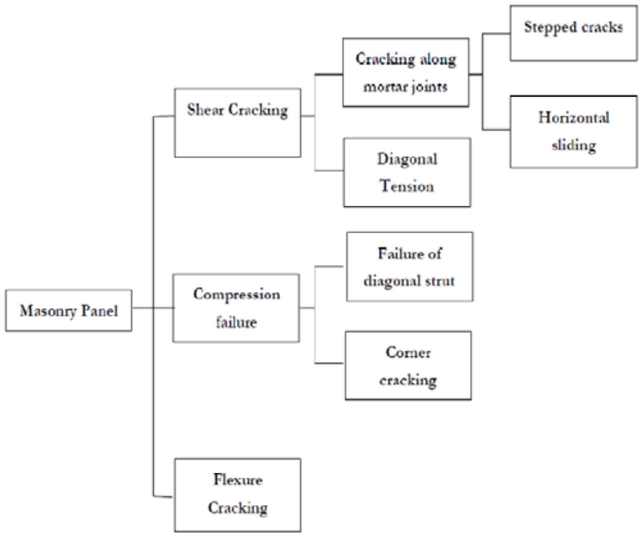
FAILURE MECHANISM

19



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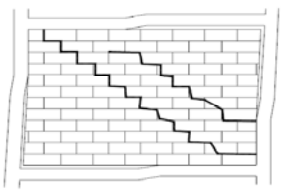
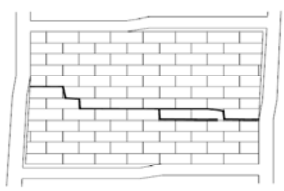
FAILURE MECHANISM (Cont…)

20

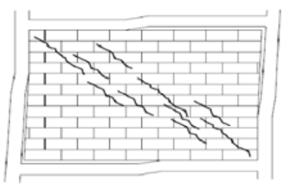
SHEAR CRACKINGS



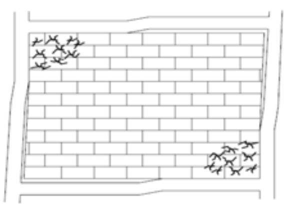
Stepped Cracking



Horizontal Sliding



Diagonal Tension



Loaded Corner

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FAILURE MECHANISM (Cont…)

21

FLEXURAL CRACKING

- In those cases where flexure effects are predominating, such as multistory infilled frames, and the columns of the frame are very weak, flexure cracks can open in the tensile side of the panel due to the low tensile strength of the masonry .

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CODAL PROVISIONS-WORLD WIDE

22

- Very few design code has provisions on RC frames with brick masonry infill.
- Building code of Albania, China, India, Nepal, Eurocode, Israel, Philippines, Egypt etc.
  - have suggested dynamic analysis of structures
  - have different fundamental time period of vibration
- FEMA 306 suggested to analyze with equivalent struts and/or dynamic analysis

BNBC GUIDELINES

23

- BNBC 1993 – No Design Guidelines for Frame with Infill Wall
- BNBC 2010 (Upcoming) – No Design Guidelines for Frame with Infill Wall
- BNBC 2010 (Upcoming) – There is guideline of designing soft storey in the open ground floor.

BNBC GUIDELINES (Cont…)

24

- BNBC 2010 (Upcoming) –
- 2.5.19.1

  Dynamic analysis of such building may be carried out incorpo rating the strength and stiffness of infill walls and inelastic deformations in the members, particularly those in the soft storey, and the members designed accordingly.

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BNBC GUIDELINES (Cont…)

25

- 2.5.19.2.

  Alternatively, the following design criteria are to be adopted after carrying out the earthquake analysis, neglecting the effect of infill walls in other storeys. The columns and beams of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads neglecting effect of infill walls.

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CNCRP-JICA GUIDELINES

26

- To evaluate the horizontal eccentricity and vertical stiffness of buildings, it is a very important matter that infill are considered or not considered.
- Infill walls are considerably effective to the seismic design of moderate earthquakes.
- The research for infill walls shall be more developed, and design of buildings with economy and safety shall be expected hereafter in Bangladesh.

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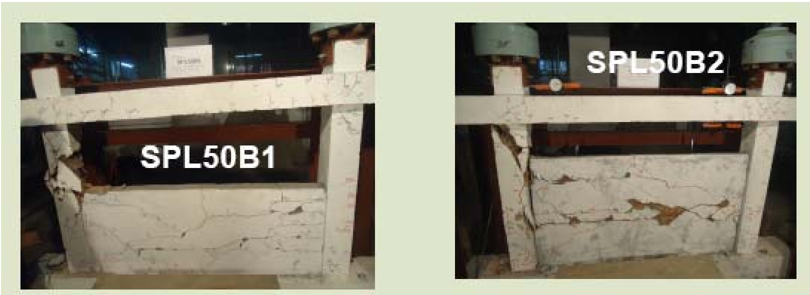
CNCRP-STRUCTURAL TEST

27

SCALED DOWN INFILLED FRAME TEST



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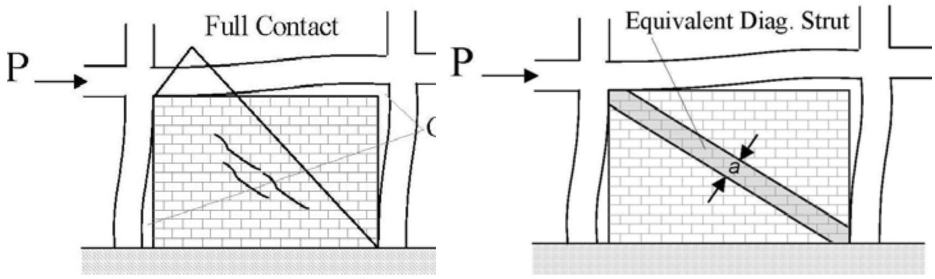
FEMA GUIDELINES-EQ. STRUT METHOD

28

- The effective width(s) of a diagonal compression strut that can be used to assess the stiffness and strength of an infill panel is initially calculated using the recommendations given in FEMA 306 (Chapter 8).
- The provisions are based on the early work of Mainstone (1971) and Mainstone and Weeks (1970) and are restated below for the convenience of the user.

FEMA GUIDELINES-EQ. STRUT METHOD

29

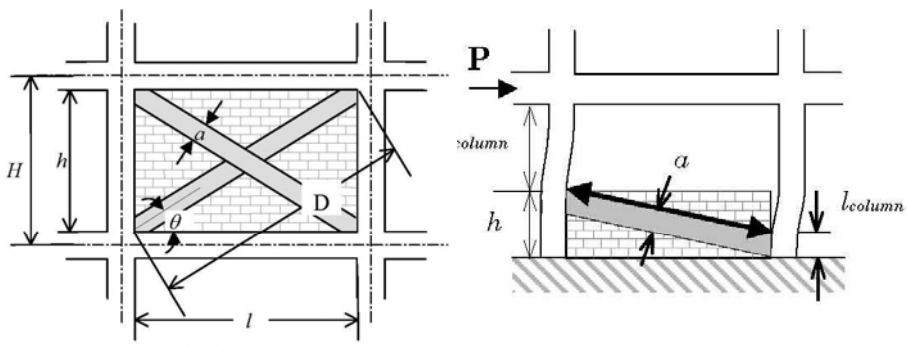


Deformation Shape

Equivalent Diagonal Strut

FEMA GUIDELINES-EQ. STRUT METHOD

30



Strut Geometry

Partially Infilled Frame

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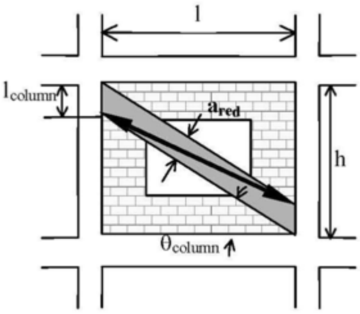
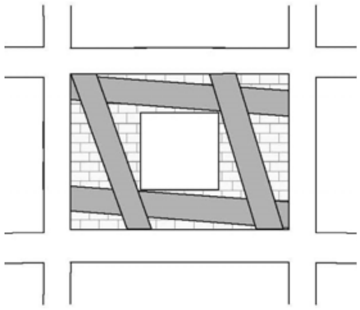
FEMA GUIDELINES-EQ. STRUT METHOD

31



Perforated Panel



Possible Strut Placement

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FEMA GUIDELINES-EQ. STRUT METHOD

32

- Thickness = Actual infill thickness (t)
- *An equivalent width, a, given by:*

$$\lambda_I H = H\left[\frac{E_m t \sin 2\theta}{4 E_c I_{col} h}\right]^{1/4}$$

By Stamford-Smith and Carter (1969)

$$a = 0.175 D(\lambda_1 H)^{-0.4}$$

By Milestone (1971)

$$a_{red} = a(R_1)_i (R_2)_i$$

$$l_{column} = \frac{a}{\cos\theta_{column}} \qquad \tan\theta_{column} = \frac{h - \dfrac{a}{\cos\theta_{column}}}{l}$$

33

FEMA GUIDELINES-EQ. STRUT METHOD

$$R_{strut} = \min\begin{Bmatrix} R_{cr} \\ R_{shear} / \cos\theta_{strut} \end{Bmatrix}$$

$$R_{cr} = a_{red}\, t_{eff}\, f'_m$$

$$R_{shear} = A_n f'_v (R_1)_i (R_2)_i$$

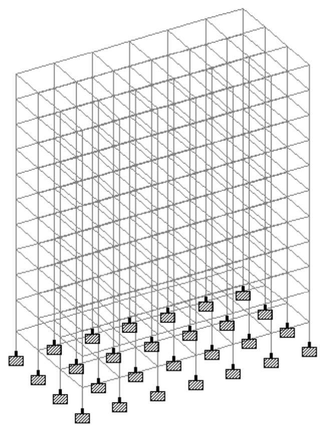
	$(R_2)_i$ for Type of Damage	
h/t	Moderate	Severe
≤ 21	0.7	0.4
> 21	Requires Repair	

34

DESIGN EXAMPLE

3D MODEL ANALYSIS

Wind = 260 km/hr
Seismic Zone 3
f'c = 3500 Psi, Ec = 3372 Ksi
f'm = 2000 Psi, Em = 1750 Ksi



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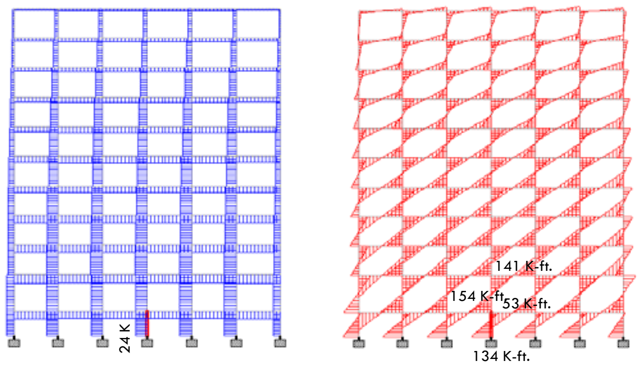
DESIGN EXAMPLE

35

- BARE FRAME ANALYSIS



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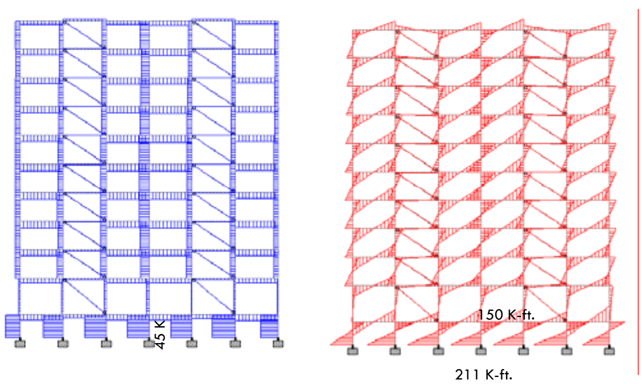
DESIGN EXAMPLE

36

- 30% STRUTS FROM GROUND FLOOR LEVEL



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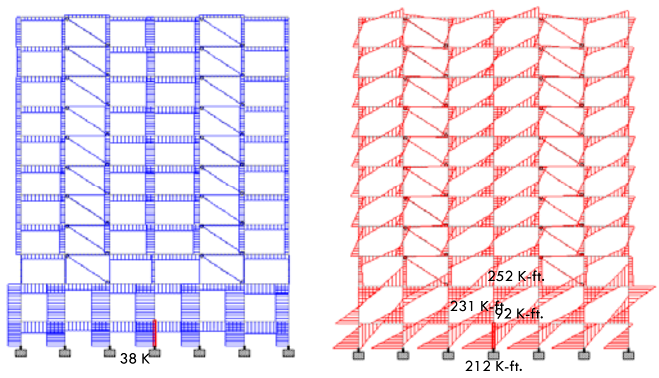
DESIGN EXAMPLE

37

- 30% STRUTS FROM FIRST FLOOR LEVEL



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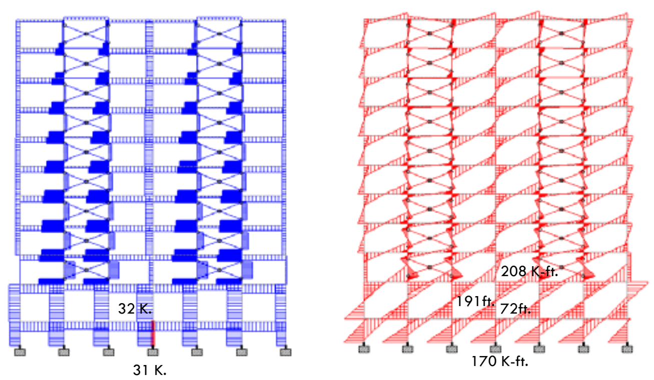
DESIGN EXAMPLE

38

- 30% STRUTS AT HINGE POINTS OF COLUMNS FROM FIRST FLOOR LEVEL



CONCLUSIONS

39

- Brick infills should be considered critically during structural design to avoid horizontal and vertical irregularities of buildings
- Analysis providing equivalent struts as per FEMA 306 may be considered
- Soft storey effect should be considered and BNBC or other guidelines appropriate must be followed
- Need to approach towards dynamic analysis

REFERENCES

40

- FEMA (Federal Emergency Management Agency) 306- Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings.
- Ghassan Al-Chaar - Evaluating Strength and Stiffness of Unreinforced Masonry Infill Structures
- Diptesh Das and C.V.R. Murty – Brick Masonry Infills in Seismic Design of RC Framed Buildings (Part I and II)
- Code Approaches to Seismic Design of Masonry-Infilled Reinforced Concrete Frames: A State-of-the-Art Review (Hemant Kaushik, Durgesh C. Rai, and Sudhir K. Jain)
- S. Haque & K M Amanat – Seismic Performance of Columns of RC Soft-Storey Buildings
- Wijanto L. S (2007) - "Seismic Assessment of Unreinforced Masonry Walls", A thesis, University of Canterbury Christchurch, New Zealand.
- Bangladesh National Building Code (BNBC 2010) - Upcoming

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THANK YOU

2nd Domestic Training

Venue: PWD Seminar Room

Duration: Apr 16, 2013 to Apr 25, 2013

Schedule

Course Title: **Short Training Course on Seismic Assessment, Retrofit Design and Construction of RC Buildings.**

Course Duration: 16/04/13 to 24/04/13

Total participant = 34

Program Schedule:

Date	Time		Title of Lecture	Resource Person
16-04-13 (Tuesday)	2:30 – 3:15		Inauguration of the course	
	3:15 – 3:30		Tea Break	
	3:30 – 5:30	A1 & A2	Basic concept on seismic evaluation of RC buildings	Md. Rafiqul Islam
17-04-13 (Wednesday)	2:30 – 3:25	A3	Overview of seismic evaluation according to Japanese Standard	Ahmed Abdullah Noor
	3:25 – 4:20	A4	Screening procedure with example (1st level)	Md. Emdadul Huq
	4:20 – 4:35		Tea Break	
	4:35 – 5:30	A5	Screening procedure with example (2nd level) [cont.]	Md. Emdadul Huq
18-04-13 (Thursday)	2:30 – 3:25	A6	Screening procedure with example (2nd level)	Md. Emdadul Huq
	3:25 – 4:20	R1	Concept on Retrofitting Design	Anup Kumar Halder
	4:20 – 4:35		Tea Break	
	4:35 – 5:30	R2	Retrofitting design methods (cont.)	Md. Mominur Rahman
21-04-13 (Sunday)	2:30 – 3:25	R3	Retrofitting design methods	Md. Mominur Rahman
	3:25 – 3:40		Tea Break	
	3:40 – 5:30	R4 & R5	Retrofitting works procedure with site visit	Md. Sohel Rahman
22-04-13 (Monday)	2:30 – 3:25	R6	Retrofitting design example of a real structure (cont.)	Anup Kumar Halder
	3:25 – 4:20	R7	Retrofitting design example of a real structure	Anup Kumar Halder
	4:20 – 4:35		Tea Break	
	4:35 – 5:30	R8	Proposed seismic demand index, "I_{so}" and others	Akira Inoue
23-04-13 (Tuesday)	2:30 – 3:25	N1	Seismic design concept for new buildings	Md. Mominur Rahman
	3:25 – 4:20	N2	Code provision for seismic analysis with example (cont.)	Md. Rafiqul Islam
	4:20 – 4:35		Tea Break	
	4:35 – 5:30	N3	Code provision for seismic analysis with example	Md. Rafiqul Islam
24-04-13 (Wednesday)	2:30 – 4:20	N4 & N5	Code provision for seismic design with example	Md. Rafiqul Islam
	4:20 – 4:35		Tea Break	
	4:35 – 5:30	N6	Effect of infill wall in seismic analysis	Md. Jahidul Islam Khan
25-042-13 (Thursday)			Closing ceremony	

List of Participants

Short Training Course on "Seismic Assessment, Retrofit Design and Construction of RC Buildings"(16-25 April,2013)

List of Participants

1	Abul Khair Mohammad Salehuddin	Deputy Secretary, MoPA
2	Md. Sayed Mahbub Morshed	Executive Engineer, PWD Design Division 1
3	Kazi Wasif Ahmad	Executive Engineer, PWD Khulna Division-1
4	Md. Mahmud Kabir	Executive Engineer, PWD Chadpur Division
5	Swarnendu Shekhar Mondal	Sub-Divisional Engineer, PWD Dhaka Div-3
6	Zahid Hasan Khan	Assistant Engineer, PWD Design Division-5
7	Mohammad Tariqul Islam	Assistant Engineer, PWD Design Division-6
8	Rashed Ahsan	Assistant Engineer, PWD Design Division-6
9	Mahmudul Hasam	Assistant Engineer, PWD Design Division-2
10	Md. Adnan Rahman	Assistant Engineer, PWD Survey Division
11	Md. Amanullah Sarkar	Assistant Engineer, PWD Maintaince Circle
12	Md. Harun-or-Rashid	Professor, Khulna University of Engineering & Technology(KUET)
13	Dr. Md. Mokhlesur Rahman	Professor, Dhaka University of Engineering & Technology(DUET)
14	Dr. Sharmim Reza Chowdhury	Associate Professor, Ahsanullah University of Science & Technology(AUST)
15	Dr. Touhidur Rahman	Associate Professor, Shahjalal University of Science & Technology(SUST)

16	Md. Tarek Hossain Khondoker	Lecturer, University of Asia Pacific
17	Md. Nuruzzaman	Assistant Engineer, Dhaka North City Corporation
18	Md. Mojaffor Uddin	Exwcutive Engineer, RAJUK
19	Engr. Boni Amin	Reserch Engineer, House Building Research Institute
20	Engr. Tapos Chowdhury	Senior Assistant Engineer, Local Government Engineering Department(LGED)
21	Md. Hossain	Executive Engineer, RAJUK
22	Shantanu Ghose Sagar	Assistant Engineer, National Housing Authority (NHA)
23	Nure Alam Siddiki	Sub-Divisional Engineer,Power Development Board (PDB)
24	Engr Sajedul Huq	Engineer, ENVIRON
25	Engr. Tanvir Quasem	Engineer, ENVIRON
26	Md. Sayeedul Haque	Engineer, BCL
27	Samy Muhammad Reza	Structural Engineer, DPM
28	kh. Mobinur Rahman	Senior Engineer, AXIS
29	M. Mahbubul Alam	Senior Engineer, CONCORD
30	Mrs. Ayesa Siddika	Engineer, STHAPATI
31	Md. Yousuf Pasha	Assistant Engineer, BEPZA

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Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

Basics of Seismic Vulnerability Assessment

Md. Rafiqul Islam
Executive Engineer
PWD Design Division – 3
&
Team Leader
Working Team – 2
CNCRP Project

CNCRP

Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

Is the Building Safe to Earthquake?

- What is the seismic intensity?
- What is the lateral load resisting system?
- What is the performance objective?
- Age of the building?
- Subsoil condition?
- Irregularity of the building?
- What is the evaluation standard?

Factors Affecting Seismic Performance

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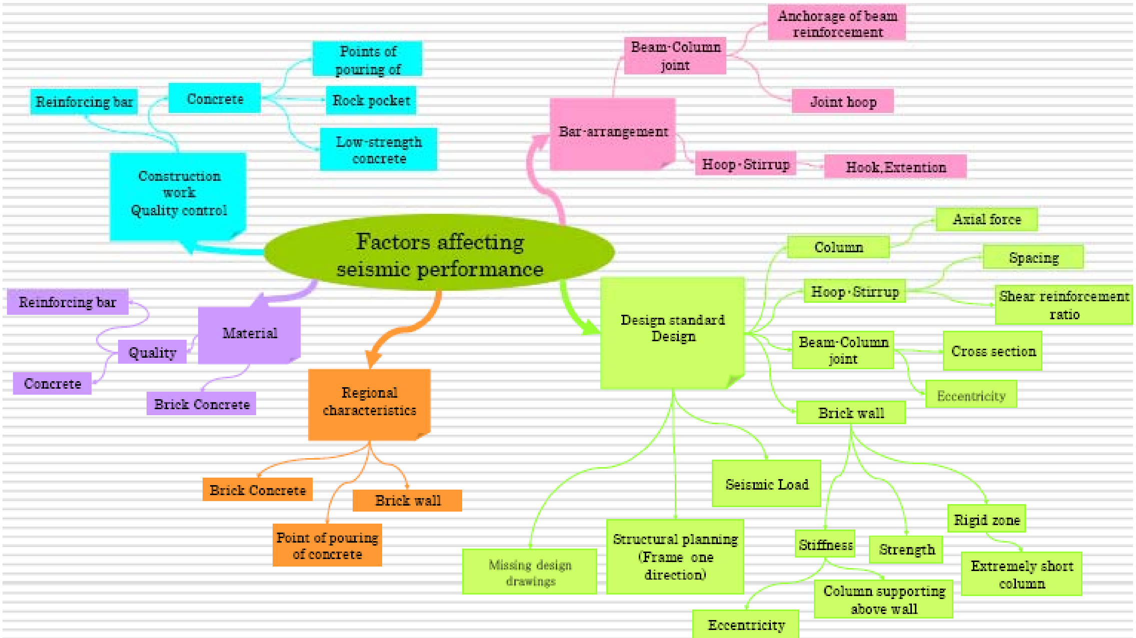
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings



Reference: Presentation 'Issues of Seismic Performance' by Yosuke Nakajima, JICA Expert Team, 2012

Selection in Performance Based Design

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

Operational | *Immediate Occupancy* | *Life Safety* | *Collapse Prevention*

Operational – negligible impact on building and it is fully operable

Immediate Occupancy – building is safe to occupy and retain its pre-earthquake strength and stiffness

Life Safety – building is safe during event but possibly not afterward

Collapse Prevention – building is on verge of collapse, probable total loss

Performance Level for Evaluation

Performance level checked for both structural and non-structural components:

1. Life Safety (LS) Performance Level:
   - Partial or total structural collapse does not occur
   - Damage to non-structural components is non-life-threatening

2. Immediate Occupancy (IO) Performance Level:
   - Vertical and lateral force resisting system retain nearly pre-earthquake strength
   - The damage is repairable while the building is occupied

Geologic site hazard and foundation hazard are also assessed.

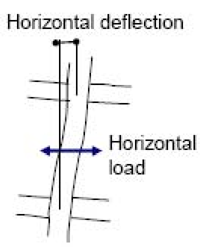
Methods of Seismic Vulnerability Assessment

1. Rapid Visual Screening (FEMA 154) [Pre evaluation sage]
2. Seismic Evaluation of Existing Building (ASCE/SEI 31-03)
   - Tier 1 – Screening phase
   - Tier 2 – Evaluation phase
   - Tier 3 – Detailed evaluation phase
3. Seismic Evaluation of Existing Reinforced Concrete Structure, 2001 [Japanese standard]
4. Euro Code 8: Part 1-4
5. Document by New Zealand Society for Earthquake Engineering
6. Report by Structural Engineering Research Centre of India

Three levels of screening in Japanese Standard

1. First level screening
   - Beam is extremely rigid and only vertical member will deform
   - Vertical members are classified into three categories
   - Concrete strength and sectional area of vertical member are required for calculation; reinforcement details is not required
2. Second level screening
   - Beam is extremely rigid and only vertical member will deform
   - Vertical members are classified into five categories
   - Reinforcement details of vertical member is required for calculation
3. Third level screening
   - Beam is flexible and hinge may form either in beam or column
   - Vertical and horizontal members are classified into eight categories
   - Calculation process is very rigorous

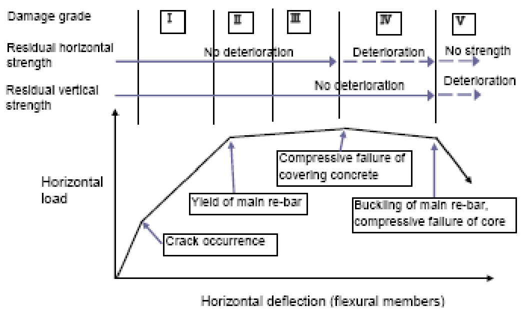
Horizontal Load-Deflection Curve of RC Columns in Japan

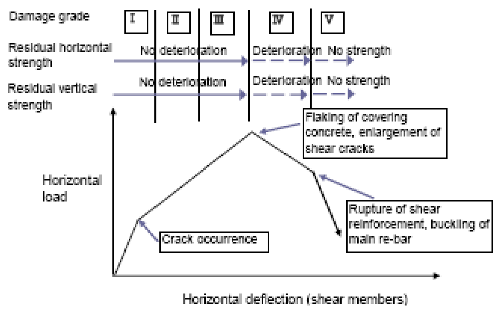


Damage grade Ⅲ

Damage grade Ⅳ

Damage grade Ⅴ

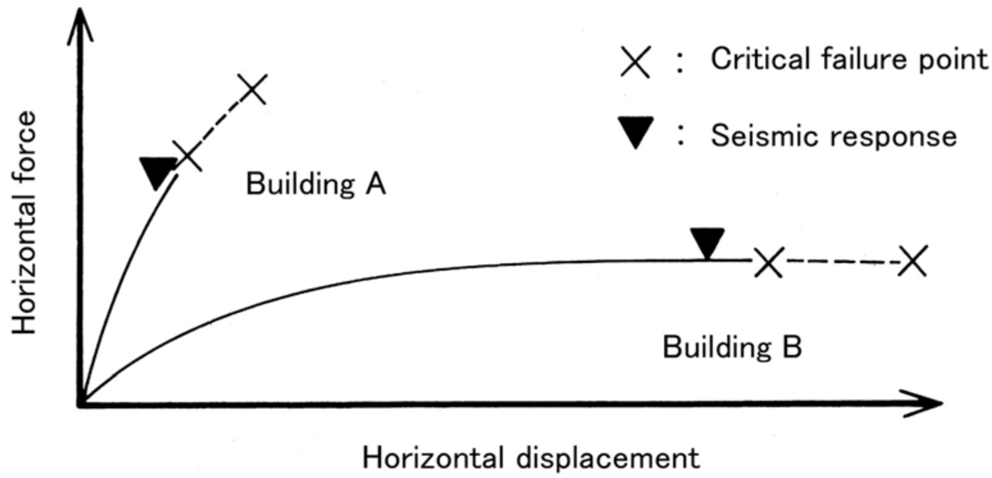




Reference: Lecture 'Seismic damages and performance of building' by Akira Inoue , JICA Expert Team, 07/06/2011

[Source: "Standard of Judgment of Damage Grade and Guidelines of Recovery Engineering for Damaged Buildings, 2001", The Japan Building Disaster Prevention Association (written in Japanese)]

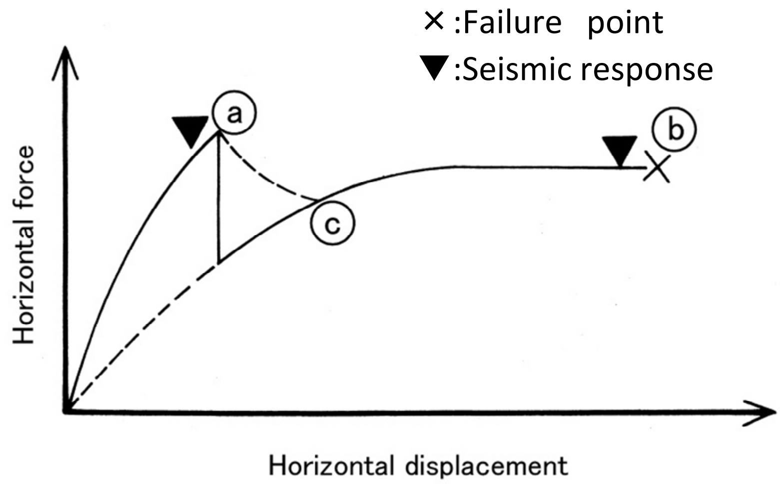
Different Types of Structure



Building 'A' resist earthquake by Strength

Building 'B' resist earthquake by Ductility

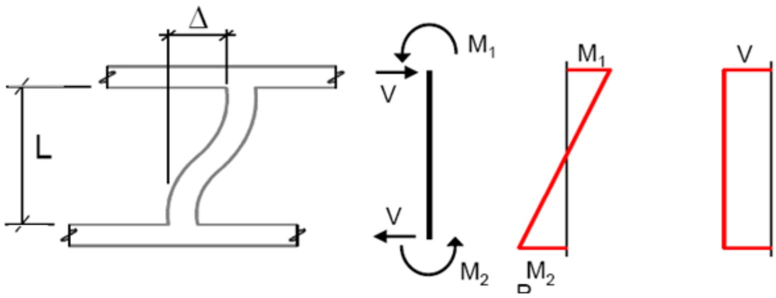
Structure Configuring Members with Different Ductility



Brittle failure of non-ductile member at 'a'
Sudden drop of stiffness from 'a' to 'c'
Performance of ductile member up to 'b'

What is Strength?

A) Flexural strength (Q_{mu}) from moment capacity of the structural member



$$Q_{mu} = \frac{M_1 + M_2}{L}$$

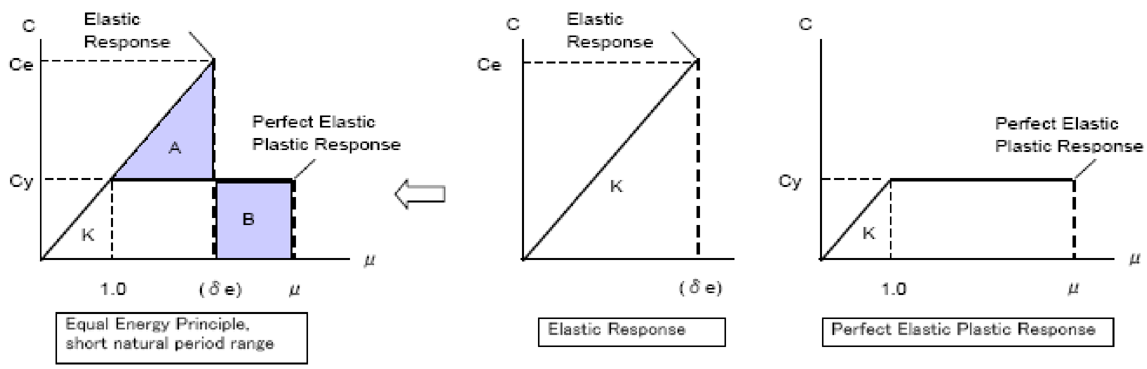
B) Shear strength (Q_{su}) from shear reinforcement of the structural member

Strength is minimum of Q_{mu} and Q_{su}

What is Ductility?

Ductility is the capacity of building material, systems or structure to absorb energy by deforming into inelastic range.

Equal Energy Principle: Ideal Non-linear Earthquake Response
In a relatively short range period building



F = Ce/ Cy =$\sqrt{2\mu - 1}$ (Newmark's formula)
Area A = Area B
C: Shear force coefficient
μ: Ductility factor

Ref: 'Seismic Design, Evaluation and Retrofitting of Building ' by Akira Inoue, JICA Expert Team

Japanese Method of Seismic Vulnerability Assessment

Seismic index of structure $I_s = E_0 \times S_D \times T$

E_0 = Basic seismic index of structure = C x F

C = Strength index $= \frac{Q}{W}$

Q = Shear strength

W = Weight on vertical member

F = Ductility index (it is function of ductility factor)

S_D = Irregularity index

T = Time index

Seismic Demand Index

Seismic demand index, $I_{so} = E_s \times Z \times G \times U$

E_s = Basic seismic demand index (depends on level of screening)

Z = Zone index (factor for seismic intensity of the site)

G = Ductility index (factor consider sub-soil condition)

U = Usage index (factor for occupancy type)

Judgment on Seismic Safety

A safe structure shall satisfy both the following checking:

A) $I_s \geq I_{so}$

I_s = Seismic index of structure

I_{so} = Seismic demand index

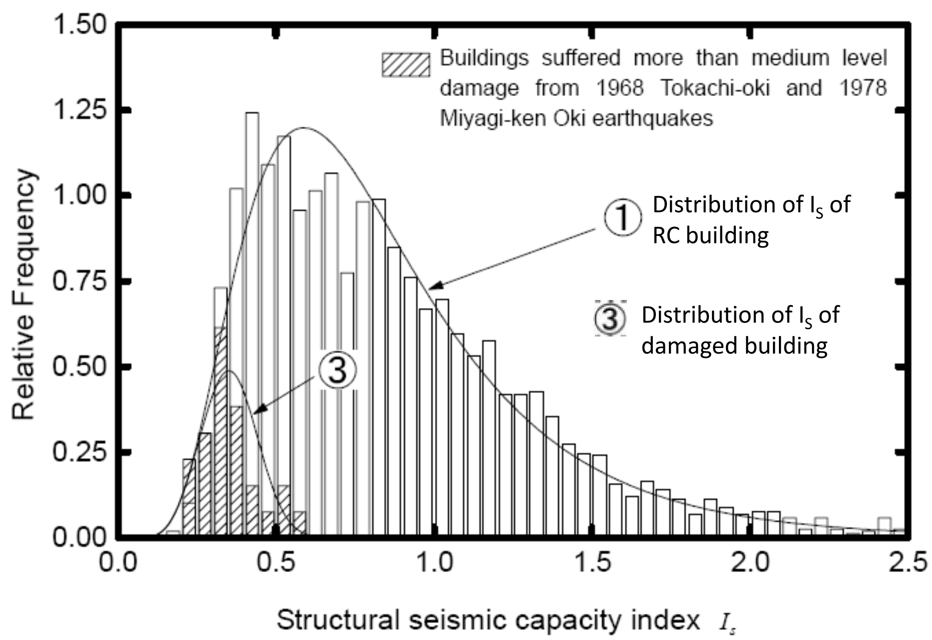
B) $C_{TU} \geq 0.3(?) \times Z \times G \times U$

C_{TU} = Cumulative strength at ultimate deformation of structure

Judgment to be applied

- Each story
- Each principal horizontal direction of a building

Study on Earthquake Damaged Buildings



Ref: Nakano, Yoshiaki and Tsuneo Okada "Reliability analysis on seismic capacity of existing reinforced concrete buildings in Japan" Journal, Transaction of Architecture Institute of Japan, No. 406, 37 – 43 (1988)

I_{SO} for Bangladeshi Buildings

Base shear, $V = \frac{ZIC}{R} W$

Z = Zone coefficient

I = Importance factor

C = Numerical coefficient for sub-soil and structural period (maximum value is 2.75)

Rearranging

$$\frac{V}{W} \times R = Z \times I \times C$$

i.e Strength index × ductility index = Z × I × C

For Dhaka Z × I × C = 0.15 × 1 × 2.75 = 0.413 (?)

Reference: Presentation 'proposed seismic demand index of structures, Iso, for existing RC buildings in Bangladesh' by Akira Inoue , JICA Expert Team, 2012

How to Calculate Strength Index (C)

Strength index, C $= \frac{Q_u}{\Sigma W}$

where ,

Q_u =Ultimate lateral load carrying capacity of vertical member

ΣW = Weight of the building supported by the story concerned

Strength index shall be modified for each story by

Story shear modification factor $= \frac{n+1}{n+i}$

n = Number of stories of a building

i = Number of story is being evaluated

How to Calculate Ductility Index (F)

Various types of deflection angle (R_{max}, R_y, R_{su}, R_{mu}, R_{mp}) of column is calculated based on:

1) Column size
2) Clear height of column
3) Axial force ratio
4) Shear force ratio
5) Tensile reinforcement ratio
6) Spacing of shear reinforcement
7) Margin against shear failure
8) etc.

How to Calculate Ductility Index (F)

A) Ductility index for shear column:

$$F = 1.0 + 0.27 \frac{R_{su} - R_{250}}{R_y - R_{250}}$$

B) Ductility index for flexural column:

(i) In case $R_{mu} < R_y$

$$F = 1.0 + 0.27 \frac{R_{mu} - R_{250}}{R_y - R_{250}}$$

(ii) In case $R_{mu} \geq R_y$

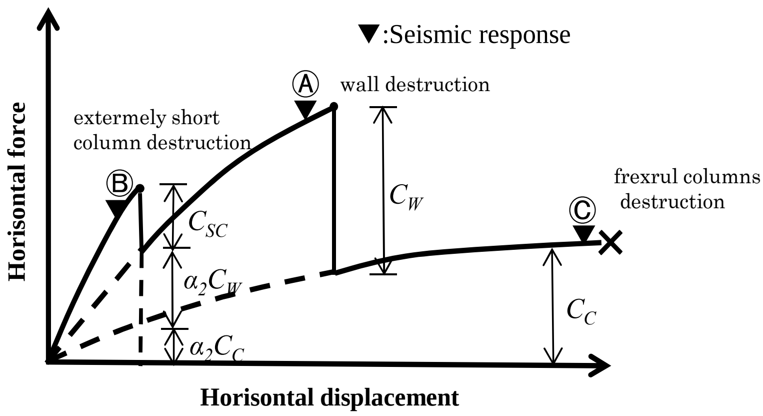
$$F = \frac{\sqrt{2\,R_{mu}/R_y - 1}}{0.75 \cdot \left(1 + 0.05\,R_{mu}/R_y\right)} \leq 3.2$$

Strength Dominant Basic Seismic Index (E_0)

$$E_0 = \frac{n+1}{n+i}\left(C_i + \sum_j \alpha_j C_j\right) \cdot F_1$$

where:

αj= Effective strength factor



Ductility Dominant Basic Seismic Index (E_0)

$$E_0 = \frac{n+1}{n+i}\sqrt{{E_1}^2 + {E_2}^2 + {E_3}^2}$$

where:

$E_1 = C_1 \times F_1$
$E_2 = C_2 \times F_2$
$E_3 = C_3 \times F_3$

C_1 = The strength index *C* of the first group (with small *F* index).
C_2 = The strength index *C* of the second group (with medium *F* index).
C_3 = The strength index *C* of the third group (with large *F* index).
F_1 = The ductility index *F* of the first group.
F_2 = The ductility index *F* of the second group.
F_3 = The ductility index *F* of the third group.

Irregularity Index (S_D)

Irregularity Index covers:

1. Regularity
2. Aspect ratio of plan
3. Expansion joint
4. Well-style area
5. Underground floor
6. Story height uniformity
7. Soft story
8. Eccentricity
9. Stiffness/mass ratio

Irregularity Index (S_D) $\leq$ 1.0

Time Index (T)

Time Index evaluates:

1. Deflection of beam and column
2. Cracking in walls
3. Fire experience
4. Occupied by chemical
5. Age of building
6. Finishing condition

Time Index (T) $\leq$ 1.0

Second-Class Prime Element

A column is a Second-Class Prime Element if

1. It fails in brittle manner
2. Its sustaining axial load can not be redistributed or not be sustained by surrounding members in the structure
3. Lateral force resisting capacity of the structure is still enough

It is necessary to check

- Shear column
- Extremely short column
- Column supporting the wall above

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

Residual Axial Load Capacity, $\eta = N/A_cF_c$

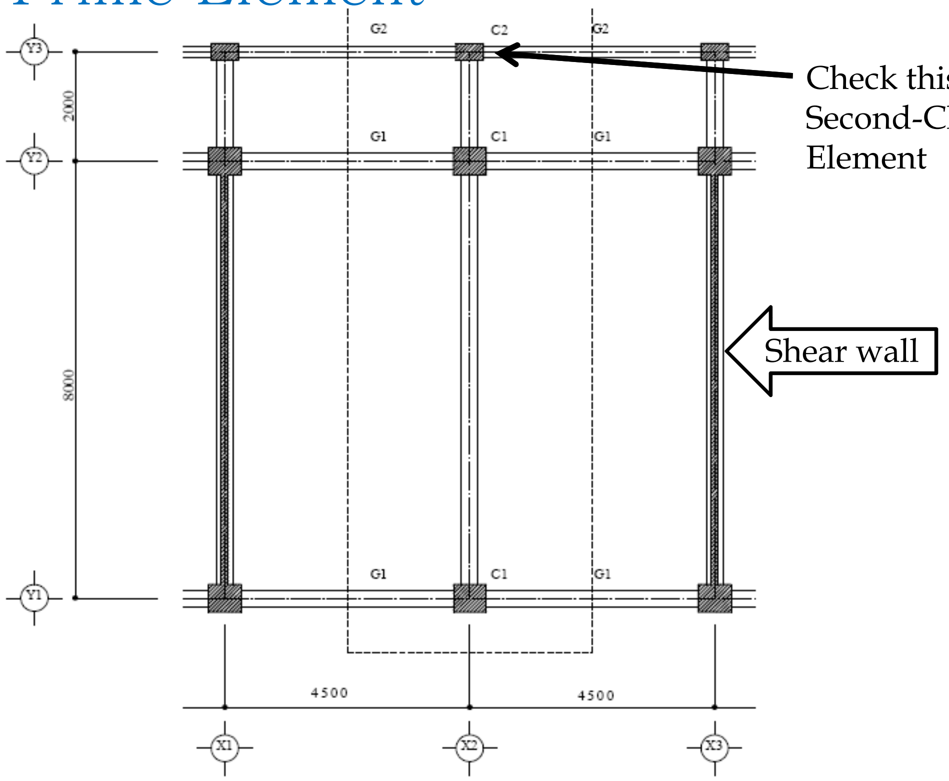
Column	p_w (%)	F=1.0	F=1.27	F=2	F=3
Extremely short column*3	$0.4 < p_w$ *1	0.4	0.3	0.1	0
	$0.2 \le p_w \le 0.4$ *2	0.3[0.4]	0.1	0	0
	$p_w < 0.2$	0[0.4]	0	0	0
Shear column	$0.4 < p_w$ *1	0.6	0.4	0.2	0
	$0.2 \le p_w \le 0.4$ *2	0.5	0.3[0.4]	0.1	0
	$p_w < 0.2$	0.4	0[0.4]	0	0
Flexural column	$0.4 < p_w$ *1	0.6	0.6	0.5	0.4
	$0.2 \le p_w \le 0.4$ *2	0.5	0.5	0.3[0.4]	0.2[0.3]
	$p_w < 0.2$	0.4	0.4	0[0.3]	0[0.2]

*1: In case that spacing is not larger than 100mm, $p_w > 0.4\%$, and sub ties are provided at the same spacing as that of main ties. In case where p_w is different in each direction, the smaller p_w can be used.

*2: In case that spacing is not larger than 100mm.

*3: The flexural column of $h_0/D \le 2$ and $F < 1.27$ is included.

Example of Second-Class Prime Element



Example of Second-Class Prime Element

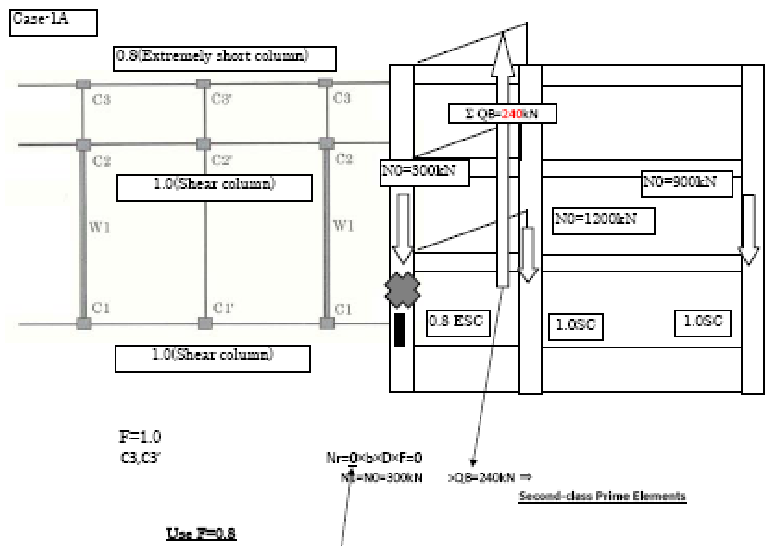


Table TN.3-1 Residual axial load capacity N_r and axial load capacity N_R
($\eta_r = N_r / A_C F_C$ [$\eta_R = N_R / A_C F_C$])
(quoted from Table 3.2.1-1 in the commentary of 3.2.1 of the Standard of 2001 Japanese version)

Column	p_w (%)	F=1.0	F=1.27	F=2	F=3
Extremely short column*3	$0.4 < p_w$ *1	0.4	0.3	0.1	0
	$0.2 \le p_w \le 0.4$ *2	0.3[0.4]	0.1	0	0
	$p_w < 0.2$	0[0.4]	0	0	0
Shear column	$0.4 < p_w$ *1	0.6	0.4	0.2	0
	$0.2 \le p_w \le 0.4$ *2	0.5	0.3[0.4]	0.1	0
	$p_w < 0.2$	0.4	0[0.4]	0	0
Flexural column	$0.4 < p_w$ *1	0.6	0.6	0.5	0.4
	$0.2 \le p_w \le 0.4$ *2	0.5	0.5	0.3[0.4]	0.2[0.3]
	$p_w < 0.2$	0.4	0.4	0[0.3]	0[0.2]

Reference: Lecture material by Yosuke Nakajima, JICA Expert Team

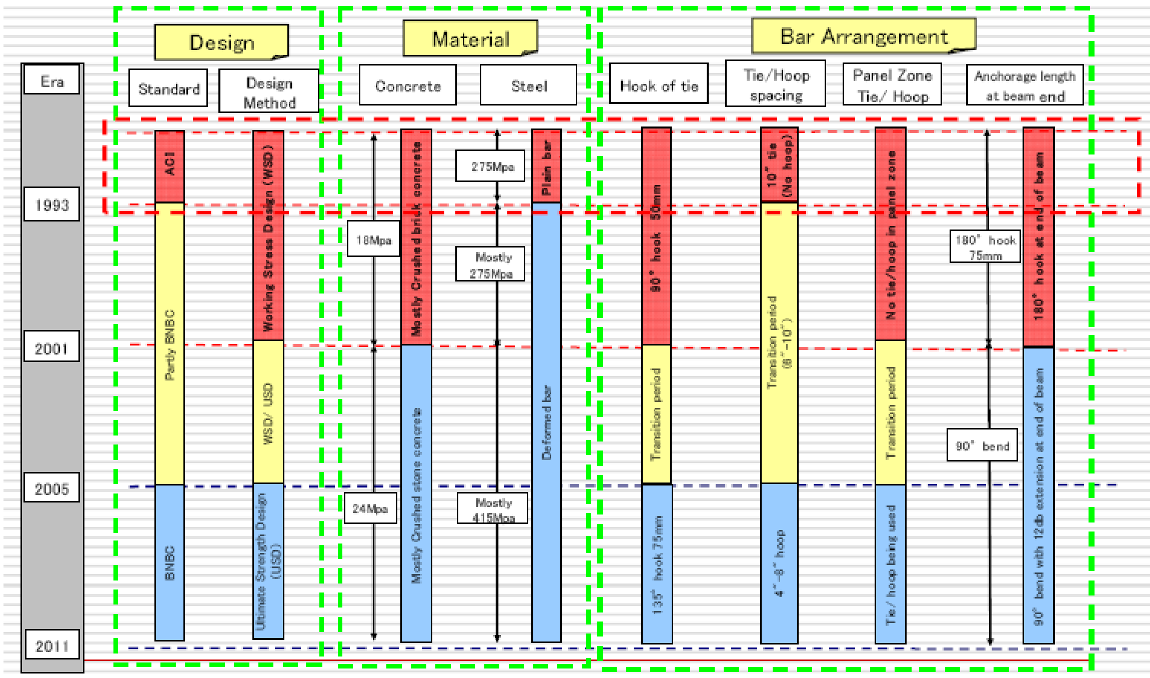
Building Inspection

Inspection Types	Inspection Objectives	Inspection Items
Preliminary inspection	To determine the applicability of the evaluation standard	Summery of the structure and building condition
Inspection without design drawings	To inspect various structural elements by conducting the actual measurement	The dimensions of building frames and reinforcing bars, arrangement of bars, etc
Detailed inspection	▪To calculate Time index and irregularity index ▪To inspect the necessity of refurbishment of aged deterioration ▪To determine the present strength related data to enhance the accuracy of evaluation procedure	Differences from original design drawings, structural cracks, deformations. Inspect material strength, concrete neutralization depth, reinforcing bar strength etc.

Benchmark for Buildings of USA

	Model Building Seismic Design Provisions							
Building Type[1, 2]	NBC[ls]	SBC[ls]	UBC[ls]	IBC[ls]	NEHRP [ls]	FEMA 178[ls]	FEMA 310[ls, io]	CBC[io]
Wood Frame, Wood Shear Panels (Type W1 & W2)	1993	1994	1976	2000	1985	*	1998	1973
Wood Frame, Wood Shear Panels (Type W1A)	*	*	1997	2000	1997	*	1998	1973
Steel Moment-Resisting Frame (Type S1 & S1A)	*	*	1994[4]	2000	**	*	1998	1995
Steel Braced Frame (Type S2 & S2A)	1993	1994	1988	2000	1991	1992	1998	1973
Light Metal Frame (Type S3)	*	*	*	2000	*	1992	1998	1973
Steel Frame w/ Concrete Shear Walls (Type S4)	1993	1994	1976	2000	1985	1992	1998	1973
Reinforced Concrete Moment-Resisting Frame (Type C1)[3]	1993	1994	1976	2000	1985	*	1998	1973
Reinforced Concrete Shear Walls (Type C2 & C2A)	1993	1994	1976	2000	1985	*	1998	1973
Steel Frame with URM Infill (Type S5, S5A)	*	*	*	2000	*	*	1998	*
Concrete Frame with URM Infill (Type C3 & C3A)	*	*	*	2000	*	*	1998	*
Tilt-up Concrete (Type PC1 & PC1A)	*	*	1997	2000	*	*	1998	*
Precast Concrete Frame (Type PC2 & PC2A)	*	*	*	2000	*	1992	1998	1973
Reinforced Masonry (Type RM1)	*	*	1997	2000	*	*	1998	*
Reinforced Masonry (Type RM2)	1993	1994	1976	2000	1985	*	1998	*
Unreinforced Masonry (Type URM)[5]	*	*	1991[6]	2000	*	1992	*	*
Unreinforced Masonry (Type URMA)	*	*	*	2000	*	*	1998	*

No Benchmark for Buildings in Bangladesh



Reference: 'Issues of Seismic Performance' by Yosuke Nakajima, JICA Expert Team

Difficulties of Seismic Assessmei of Bangladeshi Buildings

1. Missing architectural and structural design of existing building.
2. Lack of reliability in construction even if drawing is available.
3. A few or no study about lateral load resisting system of building of our country.
4. Effect of infill masonry wall in frame structure.
5. Performance of mixed type (masonry + RC frame) structure.
6. Reinforcing bars of existing structure are significantly corroded.
7. Etc.

Thank you

CNCRP

(CAPACITY DEVELOPMENT ON NATURAL DISASTER RESISTANT TECHNIQUES OF CONSTRUCTION AND RETROFITTING FOR PUBLIC BUILDINGS)

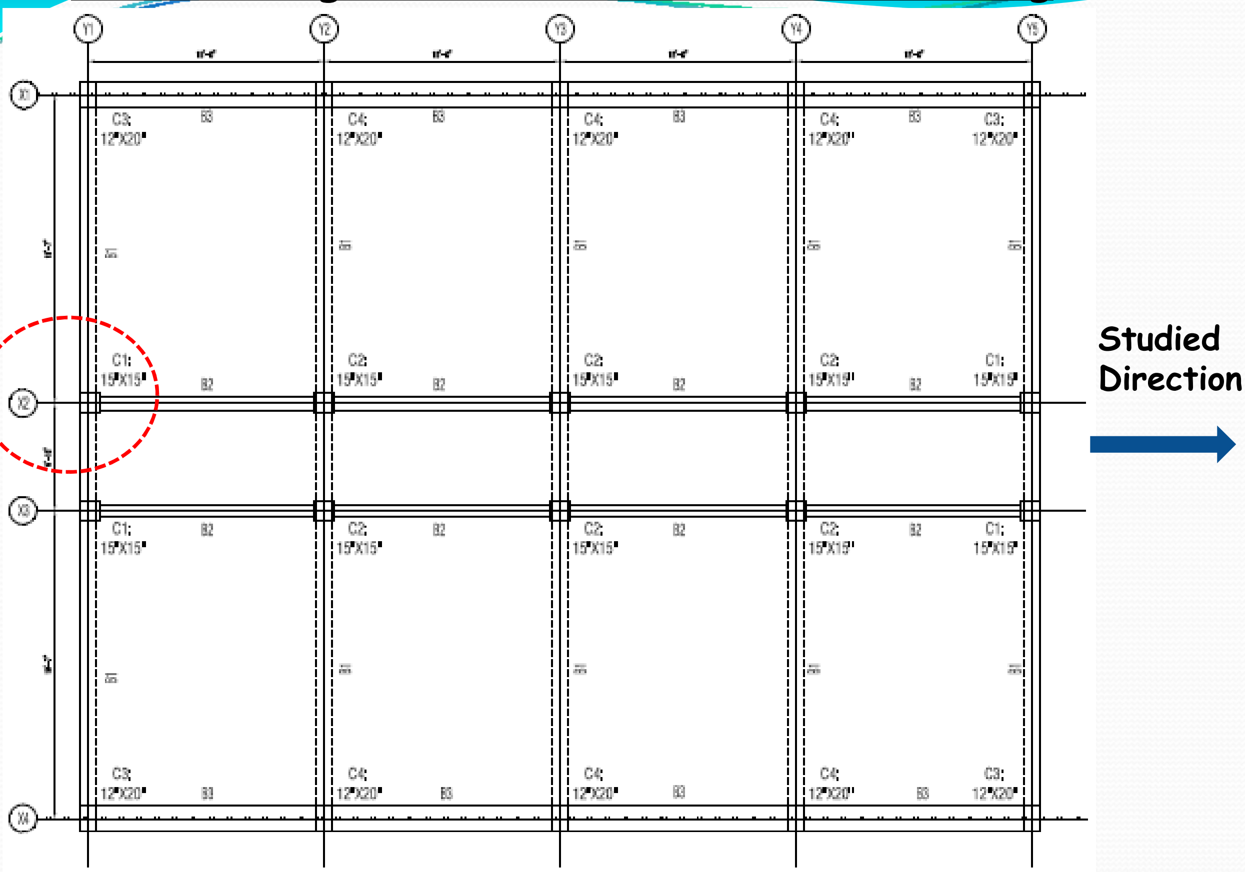
Technical co-operation project between –PWD & JICA

PRESENTATION ON

Screening Procedure with example (1st & 2nd Level)

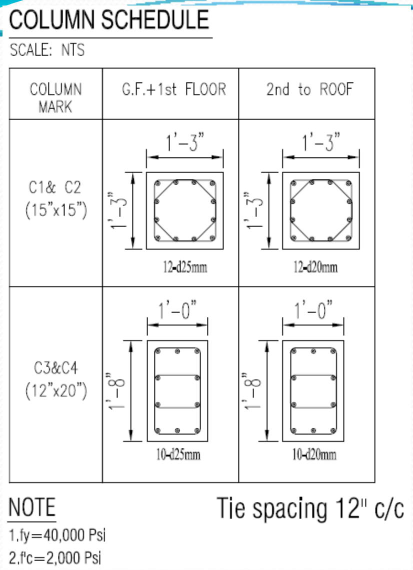
Md. Emdadul Huq
Member of Working Team-2

PLAN : Bangladesh Secretariat Clinic Building

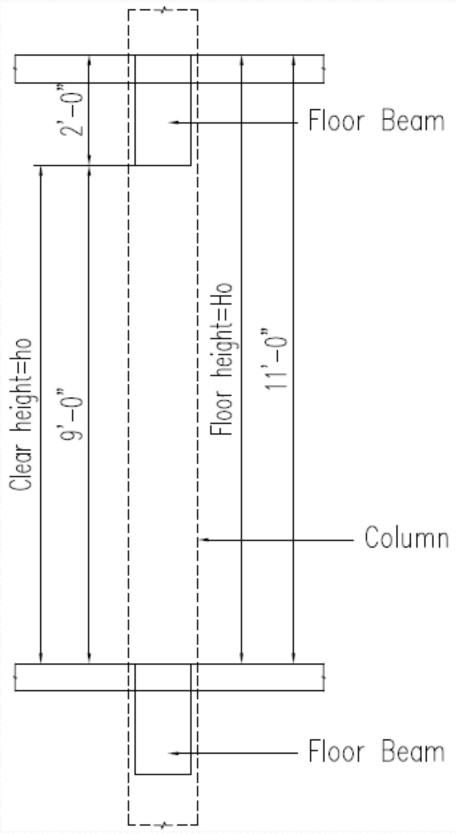


Studied Direction

Structural Drawing



BEAM MARK	SIZE
B1	12"X24"
B2	12"X22"
B3	12"X22"



About The Building

1. Name: Bangladesh Secretariat Clinic(Hospital Building)
2. Design period of the building is 1984
3. 5(Five)-Storied framed structured building.
4. Seismic detailing not provided
5. f'c = 2000 Psi = 13.79 N/mm^2
6. fy = 40000 Psi = 275 N/mm^2

Japanese Standard (Contd..)

In the **Japanese standard** three levels of seismic screening procedure.

1) 1st level screening procedure.
2) 2nd level screening procedure.
3) 3rd level screening procedure.

1st level:

-Simplest
(easy to calculation in comparison with other two evaluation procedure)
-More conservative
-Only X-sectional area & Concrete Strength of vertical Member is considered to calculate the strength
-Inelastic deformability is neglected in this level.

Japanese Standard

2nd level:

-More detail than 1st level screening procedure.
-Assuming that the strength of beam is greater than that of column(Weak column & Strong Beam)
-Evaluate ultimate strength & plastic deformation capacity of vertical members based on x-section, bar detail & material strength.

3rd level:

-Building characteristics are examined in greater detail than in the 2nd level screening procedure
-3rd level is more reliable than 2nd level screening procedure where weak beam in structure

Basic Concept of Seismic evaluation

Seismic Index of Structure (Is) = E_o X S_d x T

Where

E_o = Basic Seismic Index of Structure

= C X F

C= Strength Index

F=Ductility Index

S_d = Irregularity Index

T = Time Index

UNDERSTANDING FAILURE MODE

(STRUCTURE TYPE AND FAILURE MODE)

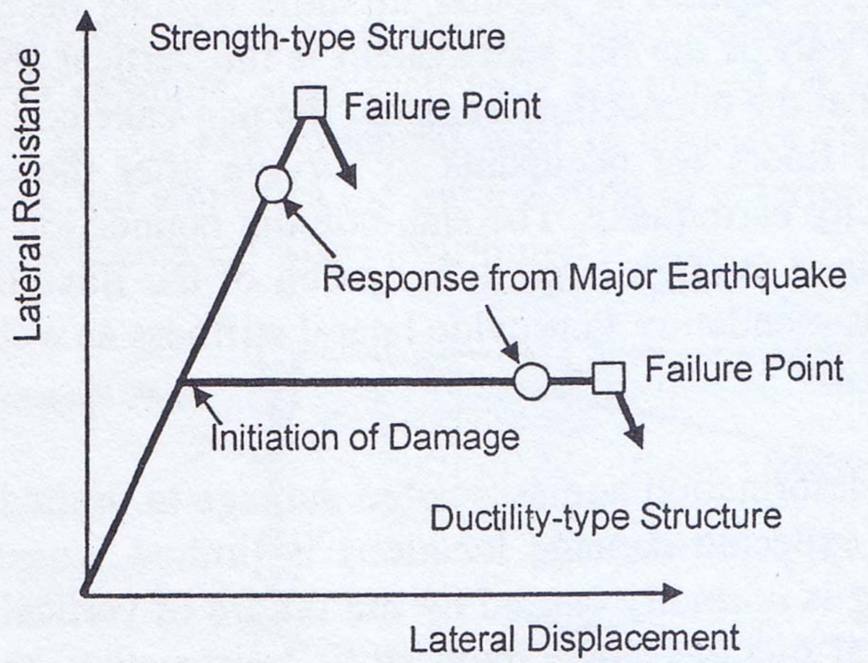
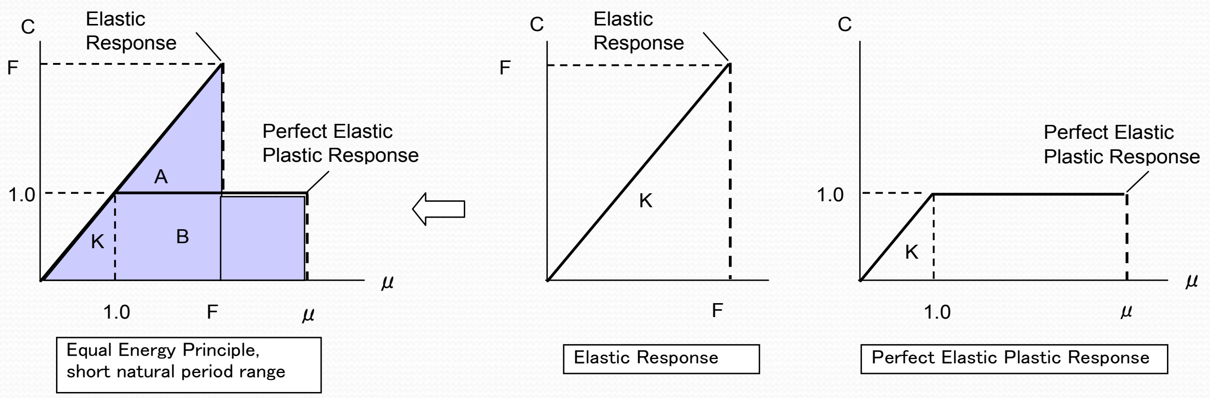


Fig. : Strength and Ductility of a Structure

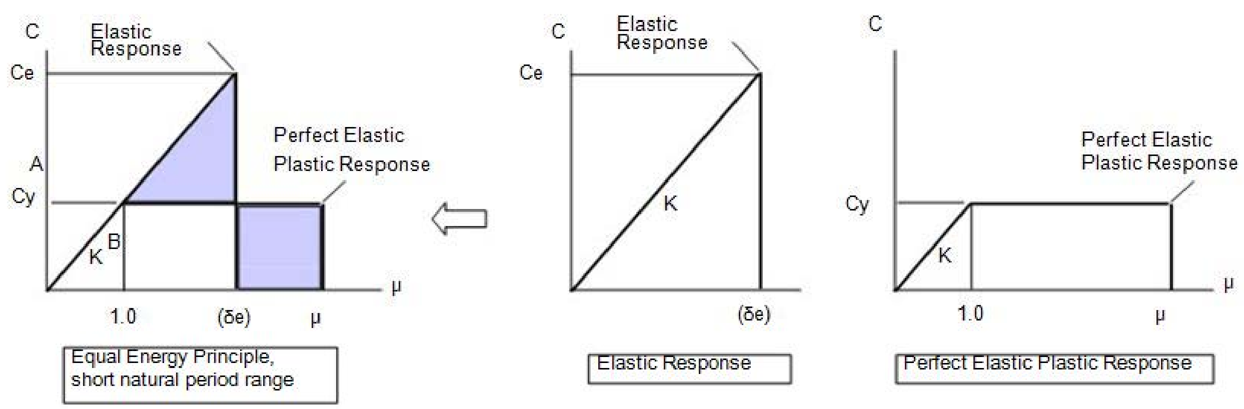
Basic Concept of Eo (Contd..)

"C x F" shows basic seismic performance of a structure. "C" is strength index, which is horizontal strength divided by building weight. "F" is ductility index. This "F" is developed based on (so called) Newmark's principle, and is related to ductility factor μ as shown below. This Equal Energy Principle for an ideal non-linear earthquake response is accepted practically in case of buildings with relatively short range natural period.



$F = R = \sqrt{2\mu - 1}$ (Newmark's formula)
Area A = Area B
μ: Ductility factor
C: Shear force coefficient

BASIC CONCEPT OF Eo (Contd..)



Cy = Min Base Shear Coefficient Structural System(Elastic Plastic Response)
Ce =Ground Motion Produces Elastic Response Base Shear(Elastic Response)
μ= Ductility(Ultimate Deformation/Yield Deformation)

$$C_e = C_y\sqrt{2\mu - 1} \quad \text{for short period systems}$$

$$C_e = C_y \cdot \mu \quad \text{for long period systems}$$

$$E_0 = C \cdot F$$

BASIC SEISMIC INDEX OF STRUCTURE (Eo) Contd…

Two Types Seismic Index

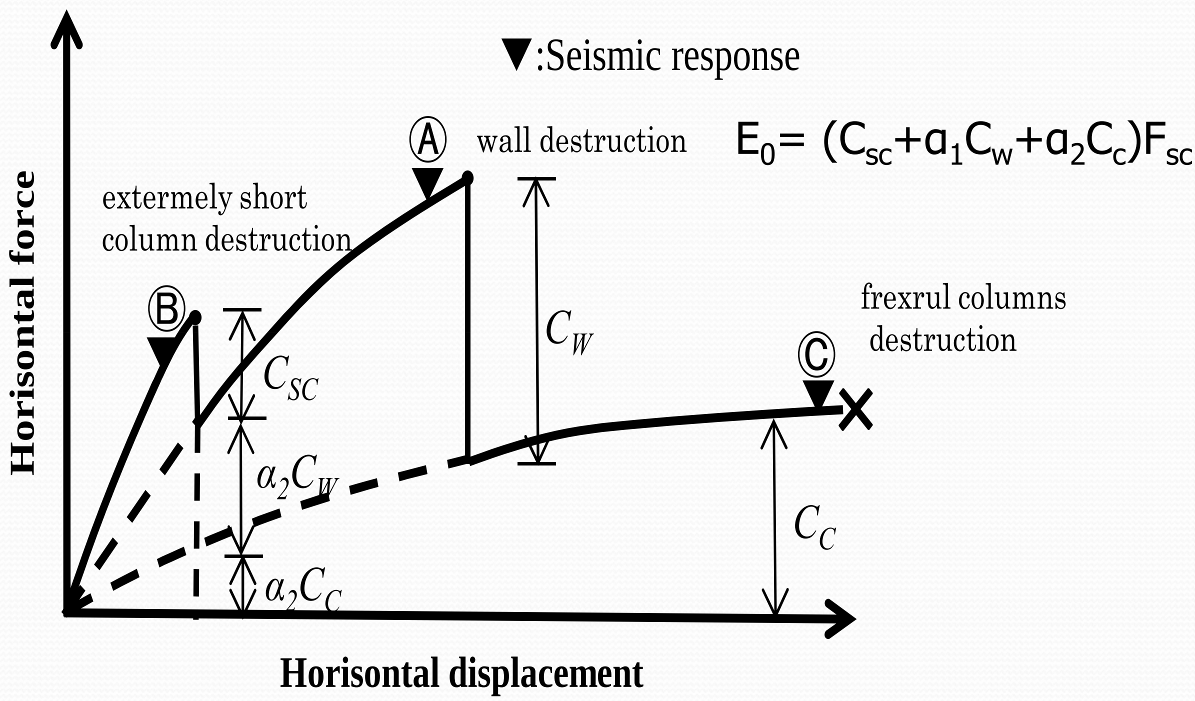
A. Strength-Dominant Structure

$$E_0 = \frac{n+1}{n+i}\left(C_1 + \sum_j \alpha_j C_j\right) \cdot F_1$$

B. Ductility-Dominant Structure

$$E_0 = \frac{n+1}{n+i}\sqrt{E_1^2 + E_2^2 + E_3^2}$$

A) Strength-Dominant Structure (Contd.)



C-F Relation Among Extremely brittle Column, Shear Wall & Flexural Column

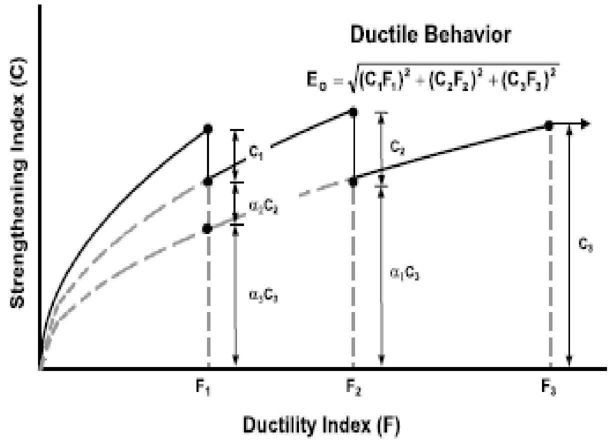
B) Ductility-Dominant Structure (Contd.)

$$E_0 = (n+1)/(n+i)\sqrt{(E_1^2 + E_2^2 + E_3^2)}$$

Where

$$E_1 = C_1 F_1$$
$$E_2 = C_2 F_2$$
$$E_3 = C_1 F_3$$

C_1 = The strength index *C* of the first group (with small *F* index).
C_2 = The strength index *C* of the second group (with medium *F* index).
C_3 = The strength index *C* of the third group (with large *F* index).
F_1 = The ductility index *F* of the first group.
F_2 = The ductility index *F* of the second group.
F_3 = The ductility index *F* of the third group.



Classification of vertical members in the 1st level screening procedure

Vertical member	Definition
Column	Columns having h_o/D larger than 2
Extremely short column	Columns having h_o/D equal to or less than 2
Wall	Walls including those without boundary columns

Ductility index in the 1st level screening

Vertical member	Ductility index *F*
Column ($h_0/D>2$)	1.0
Extremely short column ($h_0/D\leq 2$)	0.8
Wall	1.0

Note: h_o : Column clear height
D : Column depth

Basic Seismic Index of Structure(E_0) For 1st level Screening

$$E_0 = \frac{n+1}{n+i}(C_W + \alpha_1 C_C) \cdot F_W$$

$$E_0 = \frac{n+1}{n+i}(C_{SC} + \alpha_2 C_W + \alpha_3 C_C) \cdot F_{SC}$$

Where:
n = Number of stories of a building.
i = Number of the story for evaluation, where the first story is numbered as1 and the top story as n.
C_W = Strength index of the walls.
C_C = Strength index of the columns.
α_1 = Effective strength factor of the columns at the ultimate deformation of the walls, which may be taken as 0.7. The value should be 1.0 in case of C_w=0.
α_2 = Effective strength factor of the walls at the ultimate deformation of the extremely short columns, which may be taken as 0.7.
α_3= Effective strength factor of the columns at the ultimate deformation of the extremely short columns, which may be taken as 0.5.
F_W = Ductility index of the walls , which may be taken as 1.0.
F_{sc} = Ductility index of the extremely short columns, which may be taken as 0.8.

Calculation of Shear Stress & Ductility Index

ST0RY		AT GRID X1Y1
5	COLUMN	C1
	h0/D	9x12/15=7.2
	CATEGORY	COLUMN
	τ(N/mm²)	0.7
	F	1.0
4	COLUMN	C1
	h0/D	7.2
	CATEGORY	COLUMN
	τ(N/mm²)	0.7
	F	1.0
3	COLUMN	C1
	h0/D	7.2
	CATEGORY	COLUMN
	τ(N/mm²)	0.7
	F	1.0
2	COLUMN	C1
	h0/D	7.2
	CATEGORY	COLUMN
	τ(N/mm²)	0.7
	F	1.0
1	COLUMN	C1
	h0/D	7.2
	CATEGORY	COLUMN
	τ(N/mm²)	0.7
	F	1.0

Calculation of Shear Stress & Ductility Index

STORY		X2&Y2	X2&Y2	X1&Y1	X1&Y2
5	COLUMN	C1	C2	C3	C4
	h0/D	7.2	7.2	9.0	9.0
	CATEGORY	COLUMN	COLUMN	COLUMN	COLUMN
	τ(N/mm²)	0.7	0.7	0.7	0.7
	F	1.0	1.0	1.0	1.0
4	COLUMN	C1	C2	C3	C4
	h0/D	7.2	7.2	9.0	9.0
	CATEGORY	COLUMN	COLUMN	COLUMN	COLUMN
	τ(N/mm²)	0.7	0.7	0.7	0.7
	F	1.0	1.0	1.0	1.0
3	COLUMN	C1	C2	C3	C4
	h0/D	7.2	7.2	9.0	9.0
	CATEGORY	COLUMN	COLUMN	COLUMN	COLUMN
	τ(N/mm²)	0.7	0.7	0.7	0.7
	F	1.0	1.0	1.0	1.0
2	COLUMN	C1	C2	C3	C4
	h0/D	7.2	7.2	9.0	9.0
	CATEGORY	COLUMN	COLUMN	COLUMN	COLUMN
	τ(N/mm²)	0.7	0.7	0.7	0.7
	F	1.0	1.0	1.0	1.0
1	COLUMN	C1	C2	C3	C4
	h0/D	7.2	7.2	9.0	9.0
	CATEGORY	COLUMN	COLUMN	COLUMN	COLUMN
	τ(N/mm²)	0.7	0.7	0.7	0.7
	F	1.0	1.0	1.0	1.0

Calculation of column strength(C)

$$C_C = \frac{\tau_C \cdot A_C}{\Sigma W} \cdot \beta_c \qquad \beta_c = \frac{F_c}{20} \qquad F_c \le 20$$

$$C_{SC} = \frac{\tau_{SC} \cdot A_{SC}}{\Sigma W} \cdot \beta_c \qquad \beta_c = \sqrt{\frac{F_c}{20}} \qquad F_c > 20$$

C_C = Strength index of columns.

C_{SC} = Strength index of extremely short columns.

τ_C = Average shear stress at the ultimate state of columns, which may be taken as 1 N/mm^2 or 0.7 N/mm^2 in case h_0/D is larger than 6.

τ_{SC} = Average shear stress at the ultimate state of extremely short columns, which may be taken as 1.5 N/mm^2.

A_{SC} = Total cross-sectional area of extremely short columns (mm^2).

A_C = Total cross-sectional area of columns (mm^2)

F_c = Compressive strength of concrete (N/mm^2)

ΣW = Total weight (dead load plus live load for seismic calculation) supported by the story concerned

Calculation of Area Unit Weight(W)

TYPE OF LOAD	TYPICAL FLOOR	ROOF	Unit
Live Load	0.80	0.30	kN/m^2
Brick Wall	4.50	0.00	kN/m^2
Floor Finish	1.25	2.00	kN/m^2
Slab Weight	3.50	3.50	kN/m^2
SW(Column+Beam)	2.25	2.25	kN/m^2
W	12.3	8.05	kN/m^2

Calculation of Floor Weight

STORY	L(m)	B(m)	A(m^2)	∑W
5	18.6	14.54	270.44	2177
4	18.6	14.54	270.44	5504
3	18.6	14.54	270.44	8830
2	18.6	14.54	270.44	12156
1	18.6	14.54	270.44	15483

Calculation of Strength Index of Column (Cc) (Contd...)

Column ID	Story	β_c	∑W	A_c (mm^2)	τ(N/mm^2)	C_c
C1	5	0.69	2177	140625	0.7	0.031
	4	0.69	5504	140625	0.7	0.012
	3	0.69	8830	140625	0.7	0.008
	2	0.69	12156	140625	0.7	0.006
	1	0.69	15483	140625	0.7	0.004

Calculation of Strength Index of Column(Cc)

Column ID	Story	β_c	$\sum W$	A_c (mm^2)	τ(N/mm^2)	C_c
C1	5	0.69	2177	140625	0.7	0.031
	4	0.69	5504	140625	0.7	0.012
	3	0.69	8830	140625	0.7	0.008
	2	0.69	12156	140625	0.7	0.006
	1	0.69	15483	140625	0.7	0.004
C2	5	0.69	2177	140625	0.7	0.031
	4	0.69	5504	140625	0.7	0.012
	3	0.69	8830	140625	0.7	0.008
	2	0.69	12156	140625	0.7	0.006
	1	0.69	15483	140625	0.7	0.004
C3	5	0.69	2177	150000	0.7	0.033
	4	0.69	5504	150000	0.7	0.013
	3	0.69	8830	150000	0.7	0.008
	2	0.69	12156	150000	0.7	0.006
	1	0.69	15483	150000	0.7	0.005
C4	5	0.69	2177	150000	0.7	0.033
	4	0.69	5504	150000	0.7	0.013
	3	0.69	8830	150000	0.7	0.008
	2	0.69	12156	150000	0.7	0.006
	1	0.69	15483	150000	0.7	0.005

Time index *T* by the first level inspection

[A] Item to be checked	[B] Degree	[C] *T* value (check circle at relevant degree)	[D] Item to be checked for the second level inspection
Deflection	Tilting of a building or obvious uneven settlement is observed	0.7	Structural cracking and deflection
	Landfill site or former rice field	0.9	
	Deflection of beam or column is observed visually	0.9	
	No correspondence to the foregoing	1	
Cracking in walls and columns	Rain leak with rust of reinforcing bar is observed	0.8	Structural cracking and deflection
	Inclined cracking in columns is obviously observed	0.9	
	Countless cracking is observed in external wall	0.9	
	Rain leak without rust of reinforcing bar is observed	0.9	
	No correspondence to the foregoing	1	
Fire experience	Trace	0.7	Structural cracking and deflection Deterioration and aging
	Experience but traceless	0.8	
	No experience	1	
Occupation	Chemical has been used	0.8	Deterioration and aging
	No correspondence to the foregoing	1	
Age of building	30 years or older	0.8	Deterioration and aging
	20 years or older	0.9	
	19 years or less	1	
Finishing condition	Significant spalling of external finishing due to aging is observed	0.9	Deterioration and aging
	Significant spalling and deterioration of internal finishing is observed	0.9	
	No problem	1	

Calculation of Is of Building (Contd...)

Story	Column ID	Column No	C_c	Total C_c	F	α	(n+1)/(n+i)	E_o	T	S_d	I_s
5	C1	4	0.031	0.64	1.00	1.00	0.60	0.39	0.8	1	0.31
	C2	6	0.031								
	C3	4	0.033								
	C4	6	0.033								

Calculation of Is of Building

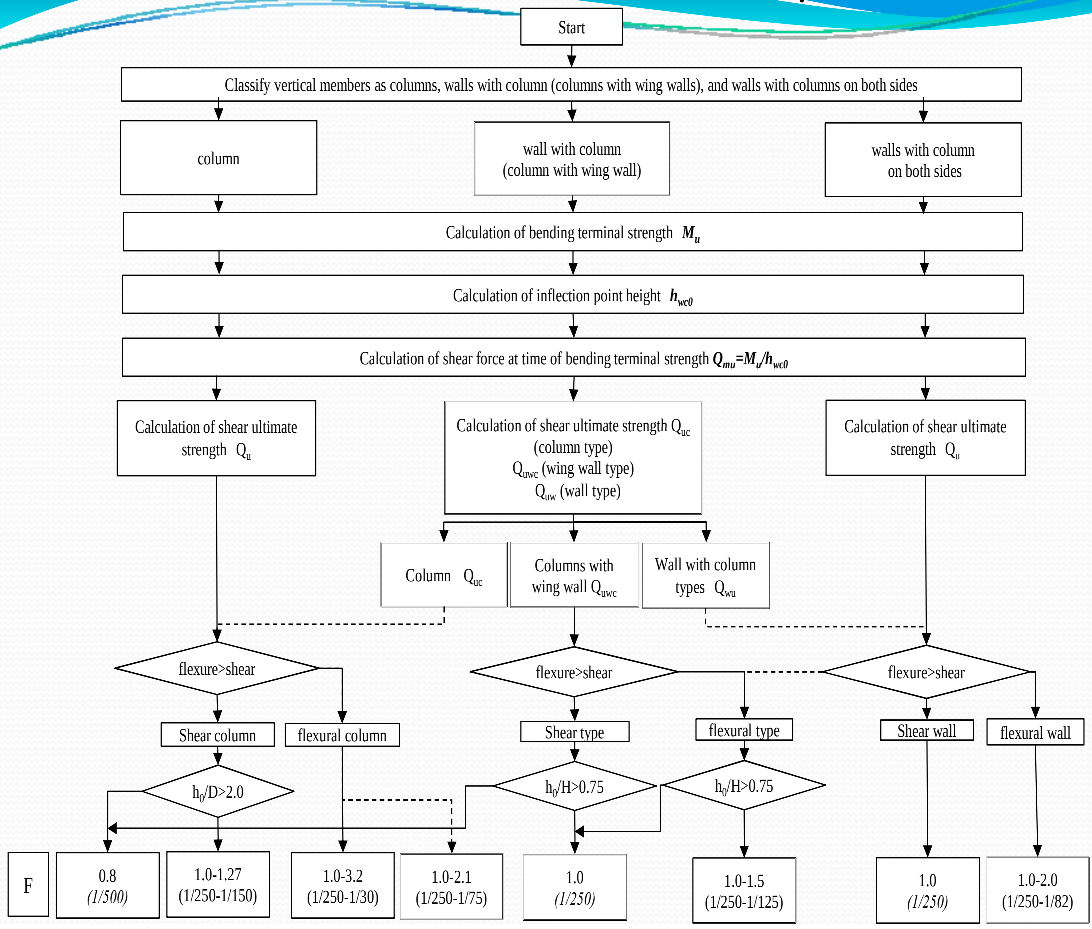
Story	Column ID	Column No	C_c	Total C_c	F	α	(n+1)/(n+i)	E_o	T	S_d	I_s
5	C1	4	0.031	0.64	1.00	1.00	0.60	0.39	0.8	1	0.31
	C2	6	0.031								
	C3	4	0.033								
	C4	6	0.033								
4	C1	4	0.012	0.25	1.00	1.00	0.67	0.17	0.8	1	0.14
	C2	6	0.012								
	C3	4	0.013								
	C4	6	0.013								
3	C1	4	0.008	0.16	1.00	1.00	0.75	0.12	0.8	1	0.10
	C2	6	0.008								
	C3	4	0.008								
	C4	6	0.008								
2	C1	4	0.006	0.12	1.00	1.00	0.86	0.10	0.8	1	0.08
	C2	6	0.006								
	C3	4	0.006								
	C4	6	0.006								
1	C1	4	0.004	0.09	1.00	1.00	1.00	0.09	0.8	1	0.07
	C2	6	0.004								
	C3	4	0.005								
	C4	6	0.005								

2^{nd} LEVEL SCREENING PROCEDURE

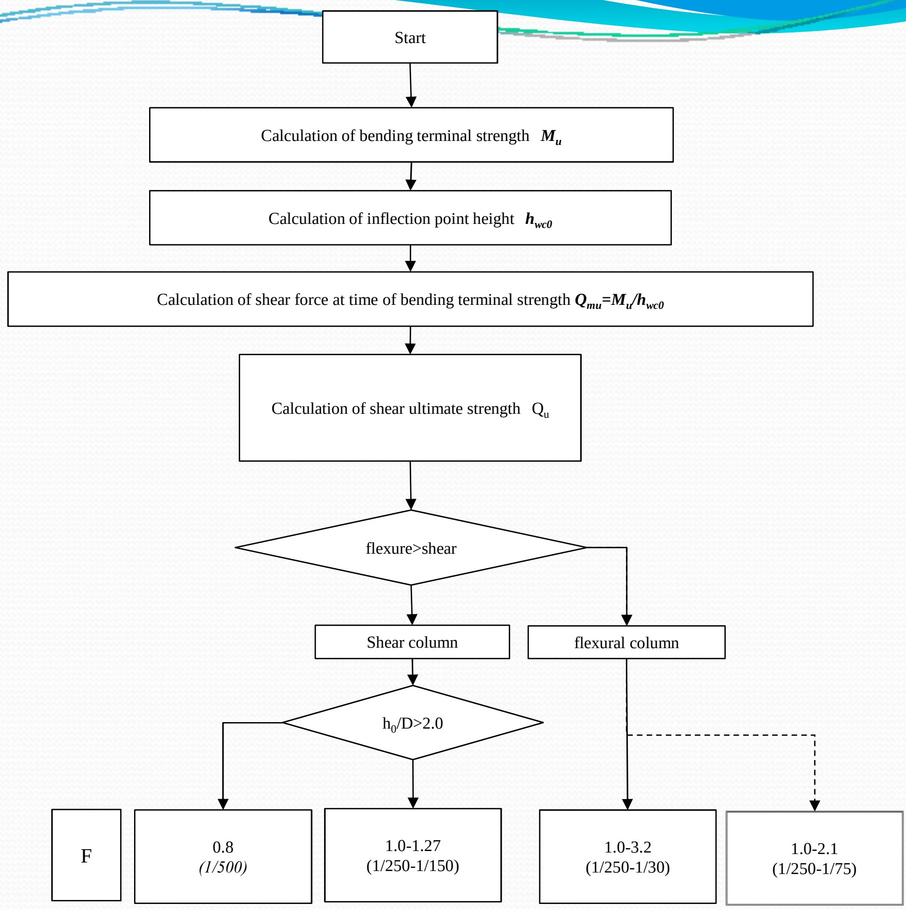
Classification of vertical members based on failure modes in the second level screening

Vertical member	Definition
Shear wall	Walls whose shear failure precede flexural
Flexural wall	Walls whose flexural yielding precede shear failure
Shear column	Columns whose shear failure precede flexural yielding, except for extremely brittle columns
Flexural column	Columns whose flexural yielding precede shear failure
Extremely brittle column	Columns whose h_0/D are equal to or smaller than 2 and shear failure precede flexural yielding

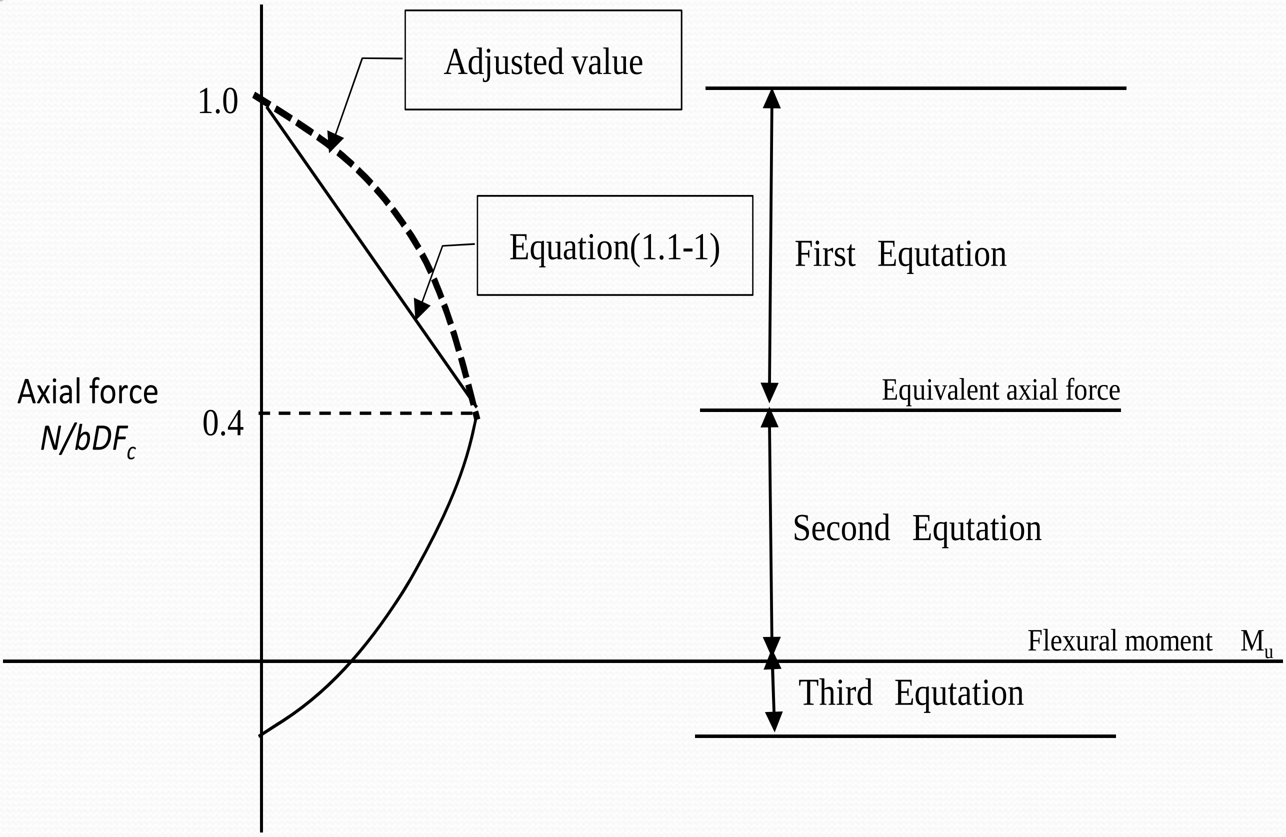
Flow Chart to Calculate The Ductility Index (Contd..)



Flow Chart to Calculate The Ductility Index



Flexural ultimate strength of columns (M_u)



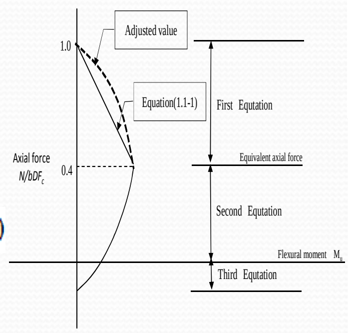
Calculation of Ultimate flexural Strength of Column(M_u)

The ultimate flexural strength of columns shall be calculated

$$M_u = \left\{0.8a_t \cdot \sigma_y \cdot D + 0.12b \cdot D^2 \cdot F_c\right\} \cdot \left(\frac{N_{max} - N}{N_{max} - 0.4b \cdot D \cdot F_c}\right) \quad For\ N_{max} \geq N > 0.4b \cdot D \cdot F_c \quad (1)$$

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.5N \cdot D \cdot \left(1 - \frac{N}{b \cdot D \cdot F_c}\right) \quad For\ 0.4b \cdot D \cdot F_c \geq N > 0 \quad (2)$$

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.4N \cdot D \quad For\ 0 > N \geq N_{min} \quad (3)$$



N_{max} = Axial compressive strength $= b \cdot D \cdot F_c + a_g \cdot \sigma_y$ (N).
N_{min} = Axial tensile strength $= -a_g \cdot \sigma_y$ (N)
N = Axial force (N)
a_t = Total cross sectional area of tensile reinforcing bars (mm^2)
a_g = Total cross sectional area of reinforcing bars (mm^2)
b = Column width (mm).
D = Column depth (mm).
σ_y = Yield strength of reinforcing bars (N/mm^2)
F_c = Compressive strength of concrete (N/mm^2)

N=79 KN

N_{max}=2976 KN

0.4bDFc=776 KN

$0.8a_t\sigma_y D$=103.8 KN.m

$0.5ND(1-N/bDF_c)$=0.5*79*375(1-0.041)/1000=14.2 KN.m

$$Q_{mu} = 2 \cdot M_u / h_0$$

Mu= 118 KN.m

Q_{mu} = 87 KN

Calculation of Column Load

COLUMN MARK	STORY	AREA(m^2)	N (KN)
C1	5	9.81	79
	4	9.81	200
	3	9.81	320
	2	9.81	441
	1	9.81	562
C2	5	18.37	148
	4	18.37	374
	3	18.37	600
	2	18.37	826
	1	18.37	1052
C3	5	7.88	63
	4	7.88	160
	3	7.88	257
	2	7.88	354
	1	7.88	451
C4	5	14.76	119
	4	14.76	300
	3	14.76	482
	2	14.76	664
	1	14.76	845

Calculation of Ultimate Shear Strength of Column(Q_{su})

$$Q_{su} = \left\{ \frac{0.053 p_t^{0.23} (18 + F_c)}{M/(Q \cdot d) + 0.12} + 0.85 \sqrt{p_w \cdot {}_s\sigma_{wy}} + 0.1\sigma_0 \right\} \cdot b \cdot j$$

Q_{su}=149 KN

p_t = Tensile reinforcement ratio (%)

p_w = Shear reinforcement ratio, p_w=0.012 for $p_w \geq 0.012$

${}_s\sigma_{wy}$ = Yield strength of shear reinforcing bars (N/mm^2)

σ_0 = Axial stress in column (N/mm^2)

d = Effective depth of column. D-50mm may be applied.

$\frac{M}{Q}$ = Shear span length. Default value is $\frac{h_0}{2}$

h_0 = Clear height of the column

j = Distance between centroids of tension and compression forces, default value is $0.8D$.

If $M/(Q \cdot d)$ is less than unity or greater than 3, the value of $M/(Q \cdot d)$ shall be unity or 3 respectively

if the value of σ_0 is greater than $8N/mm^2$, the value of σ_0 shall be $8N/mm^2$

Failure Mode Categorization According to the Strength Margin (Contd…)

Story	Column	N(KN)	Mu (KN.m)	Qmu (KN)	Qsu (KN)	Qu (KN)	Type of Column
5	C1	79	118	87	149	87	Flexural
4		200	137	102	159	102	Flexural
3		321	154	114	169	114	Flexural
2		442	226	167	185	167	Flexural
1		562	237	175	194	175	Flexural

Failure Mode Categorization According to the Strength Margin

Story	Column	N(KN)	Mu (KN.m)	Qmu (KN)	Qsu (KN)	Qu (KN)	Type of Column
5	C1	79	118	87	149	87	Flexural
4		200	137	102	159	102	Flexural
3		321	154	114	169	114	Flexural
2		442	226	167	185	167	Flexural
1		562	237	175	194	175	Flexural
5	C2	148	129	96	155	96	Flexural
4		375	160	119	173	119	Flexural
3		601	181	134	191	134	Flexural
2		828	245	181	216	181	Flexural
1		1054	224	166	234	166	Flexural
5	C3	64	92	68	145	68	Flexural
4		161	105	78	152	78	Flexural
3		258	117	87	160	87	Flexural
2		355	174	129	175	129	Flexural
1		452	183	135	183	135	Flexural
5	C4	119	100	74	149	74	Flexural
4		301	122	90	164	90	Flexural
3		483	139	103	178	103	Flexural
2		666	197	146	200	146	Flexural
1		848	202	150	214	150	Flexural

Ductility index of flexural column (F)

In case $R_{mn} < R_y$

$$F = 1.0 + 0.27 \frac{R_{mu} - R_{250}}{R_y - R_{250}}$$

In case $R_{mn} \geq R_y$

$$F = \frac{\sqrt{2R_{mu} / R_y - 1}}{0.75 \cdot (1 + 0.05 R_{mu} / R_y)} \leq 3.2$$

Where:

R_y = Yield deformation in terms of inter-story drift angle, which in principle shall be taken as R_y=1/150.

R_{250} = Standard inter-story drift angle, R_{250} = 1/250.

R_{mu} = Inter-story drift angle at the ultimate deformation capacity in flexural failure of the column.

Calculation of Upper limit of the drift angle of flexural column (${}_cR_{max}$)

Calculation of drift angle for shear force

$$ {}_cR_{\max(s)} = {}_cR_{250} \quad for \quad {}_c\tau_u / F_c > 0.2 $$

$$ {}_cR_{\max(s)} = {}_cR_{30} \quad for\ other\ case $$

${}_cR_{max(s)}$ = 1/30

$$ {}_c\tau_u = \text{Shear stress at the column strength.} $$

$$ = \min\left\{ {}_cQ_{mu}/(b\cdot j),\ {}_cQ_{su}/(b\cdot j) \right\}. $$

${}_cQ_{mu}$ = Shear force at the ultimate flexural strength of the column.

${}_cQ_{su}$ = Ultimate shear strength of the column

j = Distance between the centroids of the tension and compression forces. Default value is $0.8D$.

Calculation of drift angle for clear height

$$ {}_cR_{\max(h)} = {}_cR_{250} \quad for \quad h_o / D \le 2 $$

$$ {}_cR_{\max(h)} = {}_cR_{30} \quad for\ other\ case $$

${}_cR_{max(h)}$ = 1/30

D = Column depth.

h_o = Clear height of the column.

Calculation of drift angle for Spacing of hoops

$$ {}_cR_{\max(b)} = {}_cR_{50} \quad for \quad s / d_b > 8 $$

$$ {}_cR_{\max(b)} = {}_cR_{30} \quad for\ other\ case $$

${}_cR_{max(b)}$ = 1/50

s = Spacing of hoops.

d_b = Diameter of the flexural reinforcing bar of the column.

Calculation of drift angle for tensile reinforcement

$${}_cR_{\max(t)} = {}_cR_{250} \quad for \quad p_t > 1.0\%$$

$${}_cR_{\max(t)} = {}_cR_{30} \quad for\ other\ case$$

${}_cR_{max(t)}$= 1/30

p_t = Tensile reinforcement ratio (%).

Calculation of drift angle for axial force

$${}_cR_{\max(n)} = {}_cR_{250} \quad for \quad \eta > \eta_H$$

$${}_cR_{max(n)} = {}_cR_{30} \cdot \left(\frac{{}_cR_{250}}{{}_cR_{30}} \right)^{n'} \le {}_cR_{30} \quad for\ other\ case$$

where:

$$n' = (\eta - \eta_L)(\eta_H - \eta_L).$$

$$\eta = N_s / (b \cdot D \cdot F_c).$$

$$\eta_L = 0.25 \text{ and } \eta_H = 0.5 \text{ for } s \le 100mm.$$

$$\eta_L = 0.2 \text{ and } \eta_H = 0.4 \text{ for } s > 100mm.$$

${}_cR_{max(n)}$= 1/30

Calculation of ${}_cR_{max}$

$${}_cR_{\max} = \min\{{}_cR_{\max(n)}, {}_cR_{\max(s)}, {}_cR_{\max(t)}, {}_cR_{\max(b)}, {}_cR_{\max(h)}\}$$

${}_cR_{max(n)}$= 1/30

${}_cR_{max(s)}$= 1/30

${}_cR_{max(t)}$= 1/30

${}_cR_{max(b)}$= 1/50

${}_cR_{max(h)}$= 1/30

${}_cR_{max}$= 1/50

Yield drift angle of column($_cR_{my}$)

$$_cR_{my} = {}_cR_{150} \quad for \quad h_0 / D \geq 3.0$$

$$_cR_{my} = {}_cR_{250} \quad for \quad h_0 / D \leq 2.0$$

$_cR_{my}$ is set by interporation for $2.0 < h_0 / D < 3.0$

$_cR_{my}$ = 0.0067=1/150

$_cR_{my}$ shall not be greater than that of $_cR_{max}$

Inter story drift angle at the flexural yielding of column(R_{my})

$$R_{my} = (h_0 / H_0) \cdot {}_cR_{my} \geq R_{250}$$

where, $h_0 / H_0 \leq 1.0$

R_{my}= 0.0067=1/150

Plastic drift angle of column ($_cR_{mp}$)

$$_cR_{mp} = 10({}_cQ_{su} / {}_cQ_{mu} - q) \cdot {}_cR_{my} \geq 0$$

$$q = 1.0 \quad for \quad s \leq 100mm$$

$$q = 1.1 \quad for \quad s > 100mm$$

$_cR_{mp}$= 0.0407=1/25

$_cQ_{su}$ = Ultimate shear strength of the column.

$_cQ_{mu}$ = Shear force at the ultimate flexural strength of the column.

$_cR_{my}$ = Yield drift angle of column.

S = Spacing of hoops

Inter story drift angle at the ultimate flexural strength of column (R_{mu})

$$R_{mu} = (h_0 / H_0) \cdot {}_cR_{mu} \geq R_{250}$$

where, $h_0 / H_0 \leq 1.0$

$${}_cR_{mu} = {}_cR_{my} + {}_cR_{mp} \leq {}_cR_{30}$$

cRmy= 0.0067=1/150

cRmp= 0.0407=1/25

cRmu=cRmy+cRmp=0.0467=1/21 ≤1/30

So, cRmu = 1/30 ≤cRmax=1/50

So Final cRmax=1/50

Rmu =1/50≥1/250,So Rmu= 1/50

${}_cR_{mu}$ = Drift angle at the ultimate flexural strength of column

${}_cR_{mu}$ shall not be larger than ${}_cR_{max}$

Ductility index of flexural column (F)

1) In Case $R_{mu} < R_y$

$$F = 1.0 + 0.27 \frac{R_{mu} - R_{250}}{R_y - R_{250}}$$

F =2.59

2) In Case $R_{mu} \geq R_y$

$$F = \frac{\sqrt{2R_{mu} / R_y - 1}}{0.75 \cdot (1 + 0.05 R_{mu} / R_y)} \leq 3.2$$

Where:

R_y = Yield deformation in terms of inter-story drift angle, which in principle shall be taken as R_y=1/150.

R_{250} = Standard inter-story drift angle, R_{250} = 1/250.

R_{mu} = Inter-story drift angle at the ultimate deformation capacity in flexural failure of the column.

Calculation of Ductility Index(F) (Contd.)

Story	Column	N (KN)	Mu (KN.m)	Qmu (KN)	Qsu (KN)	Qu (KN)	Type of Column	cRmax	cRmy	cRmp	cRmu	F
5	C1	79	118	87	149	87	Flexural	50	150	25	50	2.59
4		200	137	102	159	102	Flexural	50	150	32	50	2.59
3		321	154	114	169	114	Flexural	50	150	39	50	2.59
2		442	226	167	185	167	Flexural	250	250	6604	250	1.00
1		562	237	175	194	175	Flexural	250	250	3141	250	1.00

Calculation of Ductility Index(F)

Story	Column	N (KN)	Mu (KN.m)	Qmu (KN)	Qsu (KN)	Qu (KN)	Type of Column	cRmax	cRmy	cRmp	cRmu	F
5	C1	79	118	87	149	87	Flexural	50	150	25	50	2.59
4		200	137	102	159	102	Flexural	50	150	32	50	2.59
3		321	154	114	169	114	Flexural	50	150	39	50	2.59
2		442	226	167	185	167	Flexural	250	250	6604	250	1.00
1		562	237	175	194	175	Flexural	250	250	3141	250	1.00
5	C2	148	129	96	155	96	Flexural	50	150	29	50	2.59
4		375	160	119	173	119	Flexural	50	150	42	50	2.59
3		601	181	134	191	134	Flexural	50	150	47	50	2.59
2		828	245	181	216	181	Flexural	250	250	278	250	1.00
1		1054	224	166	234	166	Flexural	250	250	82	250	1.00
5	C3	64	92	68	145	68	Flexural	50	150	15	50	2.59
4		161	105	78	152	78	Flexural	50	150	17	50	2.59
3		258	117	87	160	87	Flexural	50	150	20	50	2.59
2		355	174	129	175	129	Flexural	250	250	97	250	1.00
1		452	183	135	183	135	Flexural	250	250	100	250	1.00
5	C4	119	100	74	149	74	Flexural	50	150	16	50	2.59
4		301	122	90	164	90	Flexural	50	150	21	50	2.59
3		483	139	103	178	103	Flexural	50	150	24	50	2.59
2		666	197	146	200	146	Flexural	250	250	94	250	1.00
1		848	202	150	214	150	Flexural	250	250	76	250	1.00

Calculation of Strength Index(C)

The strength index *C* in the second level screening procedure shall be calculated by the following equation

$$C = \frac{Q_u}{\sum W}$$

C=0.16

Where:

Q_u = Ultimate lateral load-carrying capacity of the vertical members in the story concerned.

∑W= The weight of the building including live load for seismic calculation supported by the story concerned.

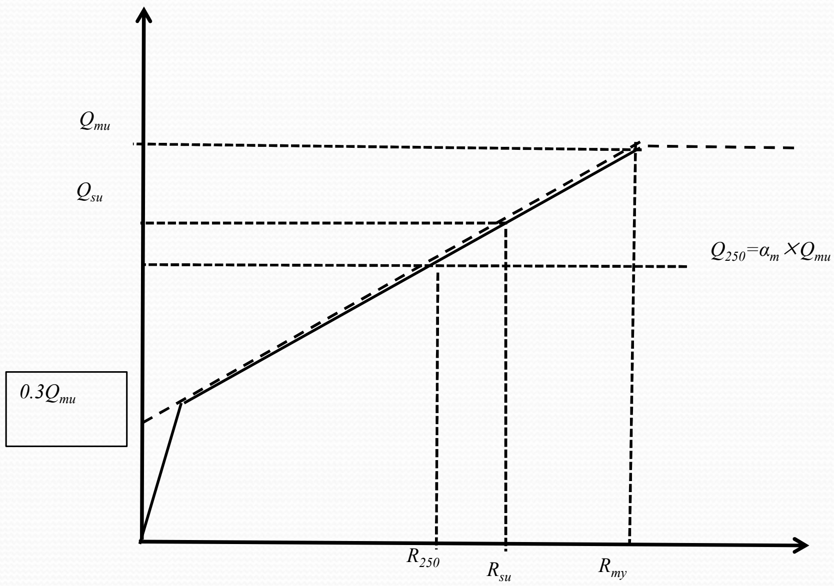
Calculation of Strength Index(C) (Contd…)

Story	Column ID	Qmu (KN)	Qsu (KN)	Qu (KN)	Column No	W (KN)	∑W (KN)	C
5	C1	87	149	87	4	79	2175	0.16
	C2	96	155	96	6	148		0.26
	C3	68	145	68	4	64		0.13
	C4	74	149	74	6	119		0.20

Calculation of Strength Index(C)

Story	Column	Qmu (KN)	Qsu (KN)	Qu (KN)	Column No	W (KN)	∑W (KN)	C
5	C1	87	149	87	4	79	2175	0.16
	C2	96	155	96	6	148		0.26
	C3	68	145	68	4	64		0.13
	C4	74	149	74	6	119		0.20
4	C1	102	159	102	4	200	5498	0.07
	C2	119	173	119	6	375		0.13
	C3	78	152	78	4	161		0.06
	C4	90	164	90	6	301		0.10
3	C1	114	169	114	4	321	8822	0.05
	C2	134	191	134	6	601		0.09
	C3	87	160	87	4	258		0.04
	C4	103	178	103	6	483		0.07
2	C1	167	185	167	4	442	12145	0.06
	C2	181	216	181	6	828		0.09
	C3	129	175	129	4	355		0.04
	C4	146	200	146	6	666		0.07
1	C1	175	194	175	4	562	15468	0.05
	C2	166	234	166	6	1054		0.06
	C3	135	183	135	4	452		0.03
	C4	150	214	150	6	848		0.06

Calculation of Effective Strength Factor(α) Contd…



Relationship between the horizontal force and the displacement

$$\alpha_m = 0.3+0.7\times R_1/R_{my}$$

α_m = Effective strength factor of a flexural column

R_{my} = Drift angle at flexural yielding.

Calculation of Effective Strength Factor(α) Contd...

Story	Column ID	F index	$1/R_{my}$	Effective Strength Factor(α)			
				R1=R500	R1=R250	R250<R1<R150	R1>R150
				F1(=0.8)	F1(=1)	F1(1<F1<1.27)	F1(1.27<=F1)
5	C1	2.59	150				1.0
	C2	2.59	150				1.0
	C3	2.59	150				1.0
	C4	2.59	150				1.0

Calculation of Effective Strength Factor(α)

Story	Column	F index	$1/R_{my}$	Effective Strength Factor(α)			
				R1=R500	R1=R250	R250<R1<R150	R1>R150
				F1(=0.8)	F1(=1)	F1(1<F1<1.27)	F1(1.27<=F1)
5	C1	2.59	150				1.0
	C2	2.59	150				1.0
	C3	2.59	150				1.0
	C4	2.59	150				1.0
4	C1	2.59	150				1.0
	C2	2.59	150				1.0
	C3	2.59	150				1.0
	C4	2.59	150				1.0
3	C1	2.59	150				1.0
	C2	2.59	150				1.0
	C3	2.59	150				1.0
	C4	2.59	150				1.0
2	C1	1.00	250		1.0		
	C2	1.00	250		1.0		
	C3	1.00	250		1.0		
	C4	1.00	250		1.0		
1	C1	1.00	250		1.0		
	C2	1.00	250		1.0		
	C3	1.00	250		1.0		
	C4	1.00	250		1.0		

Calculation of C_{TU} Indices

				Effective Strength Factor(α)						
				R1=R500	R1=R250	R250<R1<R150	R1>R150			
Story	**Column**	**C**	**F**	**F1(=0.8)**	**F1(=1)**	**F1(1<F1<1.27)**	**F1(1.27<=F1)**	**C_{TU}**	**C_{TU}Sd**	**Evaluation**
5	C1	0.16	2.59				1.0	0.75	0.75	OK
	C2	0.26	2.59				1.0			
	C3	0.13	2.59				1.0			
	C4	0.20	2.59				1.0			
4	C1	0.07	2.59				1.0	0.36	0.36	OK
	C2	0.13	2.59				1.0			
	C3	0.06	2.59				1.0			
	C4	0.10	2.59				1.0			
3	C1	0.05	2.59				1.0	0.25	0.25	NG
	C2	0.09	2.59				1.0			
	C3	0.04	2.59				1.0			
	C4	0.07	2.59				1.0			
2	C1	0.06	1.00		1.0			0.26	0.26	NG
	C2	0.09	1.00		1.0					
	C3	0.04	1.00		1.0					
	C4	0.07	1.00		1.0					
1	C1	0.05	1.00		1.0			0.20	0.20	NG
	C2	0.06	1.00		1.0					
	C3	0.03	1.00		1.0					
	C4	0.06	1.00		1.0					

Calculation of Eo (Contd…)

				Effective Strength Factor(α)					
				R1=R500	R1=R250	R250<R1<R150	R1>R150		
Story	**Column**	**C**	**F index**	**F1(=0.8)**	**F1(=1)**	**F1(1<F1<1.27)**	**F1(1.27<=F1)**	E_o(Eq1)	E_o(Eq2)
5	C1	0.16	2.59				1.0	1.95	1.01
	C2	0.26	2.59				1.0		
	C3	0.13	2.59				1.0		
	C4	0.20	2.59				1.0		

E_o(Eq1)=(C1+∑α1.C1)*F1

E_o(Eq2)=Sqrt((C1*F1)^2+….+(Ci*Fi)^2))

Calculation of Eo

				Effective Strength Factor(α)				Eo	
				R1=R500	R1=R250	R250<R1<R150	R1>R150		
Story	Column	C	F	F1(=0.8)	F1(=1)	F1(1<F1<1.27)	F1(1.27<=F1)	E_o(Eq1)	E_o(Eq2)
5	C1	0.16	2.59				1.0	1.95	1.01
	C2	0.26	2.59				1.0		
	C3	0.13	2.59				1.0		
	C4	0.20	2.59				1.0		
4	C1	0.07	2.59				1.0	0.93	0.49
	C2	0.13	2.59				1.0		
	C3	0.06	2.59				1.0		
	C4	0.10	2.59				1.0		
3	C1	0.05	2.59				1.0	0.65	0.34
	C2	0.09	2.59				1.0		
	C3	0.04	2.59				1.0		
	C4	0.07	2.59				1.0		
2	C1	0.06	1.00		1.0			0.26	0.13
	C2	0.09	1.00		1.0				
	C3	0.04	1.00		1.0				
	C4	0.07	1.00		1.0				
1	C1	0.05	1.00		1.0			0.20	0.10
	C2	0.06	1.00		1.0				
	C3	0.03	1.00		1.0				
	C4	0.06	1.00		1.0				

E_o(Eq1)=(C1+∑α1.C1)*F1

E_o(Eq2)=Sqrt((C1*F1)^2+….+(Ci*Fi)^2))

Calculation of Is(Contd…)

Story	Column	(n+1)/(n+i)	E_0(Eq1)	E_0(Eq2)	E_0	S_d	T	I_s
5	C1	0.60	1.95	1.01	1.95	1	0.8	0.94
	C2							
	C3							
	C4							

Calculation of Is

Story	Column	(n+1)/(n+i)	E_0(Eq1)	E_0(Eq2)	E_0	S_d	T	I_s
5	C1 C2 C3 C4	0.60	1.95	1.01	1.95	1	0.8	0.94
4	C1 C2 C3 C4	0.67	0.93	0.49	0.93	1	0.8	0.50
3	C1 C2 C3 C4	0.75	0.65	0.34	0.65	1	0.8	0.39
2	C1 C2 C3 C4	0.86	0.26	0.13	0.26	1	0.8	0.18
1	C1 C2 C3 C4	1.00	0.20	0.10	0.20	1	0.8	0.16

Overview of Seismic Capacity Evaluation According to Japanese Standard

February 12, 2013

By

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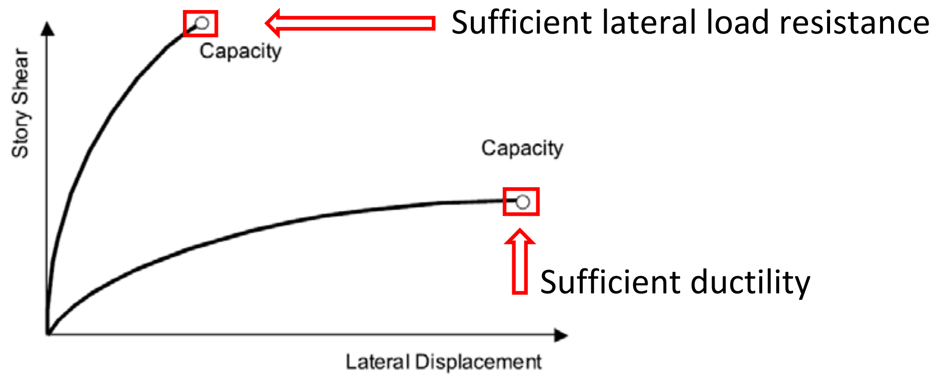
Public Works Department (PWD), Bangladesh.

Basic concept of Seismic Resistance:

A structure can resist strong ground motion without collapse if the structure is provided with

- Sufficient **lateral load resistance**
- Limited lateral load resistance with sufficient **ductility.**



Story Shear- Story Drift Relationship

Basic Concept of Seismic evaluation in Japanese Standard

The seismic index of structure *Is*,

$$Is = E_0 * S_D * T$$

Where E_0= Basic seismic index of structure

S_D= Irregularity index

T= Time index

Is should be calculated at each story and in each principal horizontal direction

Basic seismic index of structure E_0

$$E_0 = \phi * C * F$$

Where ϕ= Story Index

C = Strength Index

F = Ductility index

Basic Concept of Seismic Evaluation (Contd.)

The standard consists of **three** different level procedures: first, second and third level procedures.

The first level procedure is the simplest and most conservative procedure. Two major things are considered in this level to calculate the strength:

- Sectional area of columns and walls
- Strength of concrete.

→ Inelastic deformability is neglected in this level.

→ First Level screening should not be used if large eccentricity exists in a floor

Second and Third Level Screening ultimate lateral load carrying capacity of vertical members or frames are evaluated using

material and sectional properties together with

reinforcing details

Characteristics of the ground motion

The characteristics of the **ground motion** based on **response spectrum** is expressed by required **Seismic Capacity Index** of Structure, Iso

$$Iso= Es*Z*G*U$$

Where:

Es= Basic Seismic Demand Index of structure

Z = Zone index (Seismic activity at construction area)

G = Ground index (Amplification of ground motion by surface soil deposit)

U = Usage index. (Here in this case 1.5 is used assuming the building will be used as a shelter after a severe earthquake)

First Level Screening Procedure

Lateral strength of a story is crudely evaluated by examining the **shear strength** of columns and walls by their cross-sectional areas. The strength of girder is not examined at this stage because:

- The column is believed to be more vulnerable to earthquake force.
- Failures of columns lead to the collapse of the building.
- The girder is believed to be more ductile.

Important Features of First Level Screening

- In order to meet the high seismic design forces, the **Japanese** buildings are provided with **large columns and walls.**
- Importance of **shear wall** is also emphasized in design.

Therefore a Japanese building is believed to possess **lateral strength larger than required by code.**

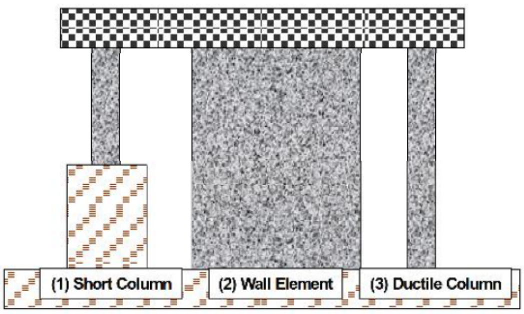
First level screening procedure is to identify these **strong** buildings by a simple calculation.

Vertical Members in First Level Screening

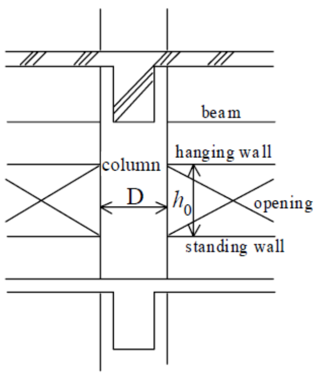
Classification of vertical members in the first level screening

Vertical member	Definition
Column	Columns having *ho/D* larger than 2
Extremely short column	Columns having *ho/D* equal to or less than 2
Wall	Walls including those without boundary columns

Where ***ho*** **: Column clear height**
D **: Column depth**



Idealized building story



Clear height and depth of column

Basic parameters used in First Level Screening

9

A crude and conservative estimation of **shear strength** per unit sectional area is used for

- short columns 1.5 Mpa
- columns 1.0 Mpa
- walls with boundary columns on both sides 3 Mpa
- walls with boundary columns on one side 2 Mpa
- walls with boundary columns with no boundary 1 Mpa

Based on dimension, materials, reinforcement ratio commonly used in Reinforced Concrete buildings in **Japan.**

Basic Seismic Index for First Level Screening

10

Short columns are likely to fail in brittle shear mode, and a small ductility index (F=0.8) is assigned.

The **wall and columns** are **assumed** to develop **70% and 50% of their strength**, respectively when the short column fails in shear.

Structural Index E_{0i} of the i story is evaluated by the following equation at the failure of short column:

$$E_{0i}=\underbrace{((n+1)/(n+i))}_{\phi} *\underbrace{(Csc +0.7*Cw +0.5*Cc)}_{C}*\underbrace{0.8}_{F}$$

Basic Seismic Index for First Level Screening (Contd.)

If short column **doesn`t exist** in a story **or** if the **failure of short column** will not lead to the **collapse** of the story, Structural Index E_{0i} of the i story is evaluated by the following equation at the failure of wall:

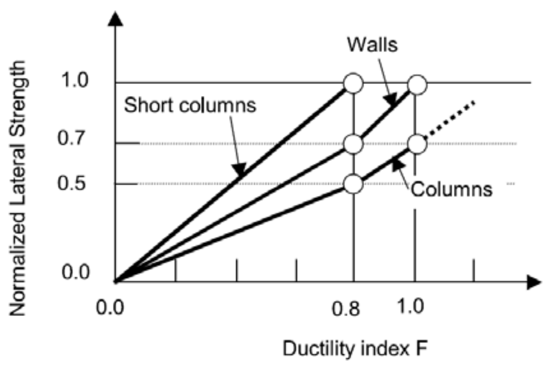
$$E_{0i} = (n+1)/(n+i) (Cw + 0.7*Cc)*1.0$$

Where the **ductility index** for wall is selected to be **1.0 and 70% of column strength** is **assumed** to be developed at the failure of wall

Basic Seismic Index for First Level Screening (Contd.)

- If no structural wall/ short column exists in a story (Cw=0), then the structural index is estimated by the following equation:

$$E_{0i} = (n+1)/(n+i)\ Cc*1.0$$



Strength and deformation relation in first level screening procedure

Irregularity/ Configuration Index

13

Irregularity/ Configuration Index S_D considers the following things:

- Irregularity in plan
- Longitudinal to transverse plan length ratio
- Expansion joints
- Existence of basement
- Abrupt discontinuity of stiffness along the height; especially soft story

A simple grading chart is provided to determine the configuration index which varies from **0.42 to 1.2**

Time/Age Index

14

In evaluating age index T the following things are to be considered:

- Observed deformation in the building caused by uneven settlement of foundation
- Cracks in columns and walls
- Rust on reinforcement
- Past and present use of chemicals
- Past fire experience
- Finishing condition and building age

Age Index T varies from **0.7 to 1.0**

Second Level Screening Procedure

15

- The combination of **different ductility levels** and shear resistance of vertical members are considered in earthquake resistance of a structure.
- The **shear resistance** of vertical members (columns and walls) must be calculated on the basis of member geometry, the amount of longitudinal and lateral reinforcement and concrete strength.
- **Failure mode**, either shear or flexural is determined by comparing shear strength and flexural strength

Classification of Vertical Members

16

- **Classification of vertical members based on failure modes in the second level screening procedure**

Vertical member	Definition	Ductility Index, F
Shear wall	Walls whose shear failure precede flexural yielding	1.0
Flexural wall	Walls whose flexural yielding precede shear failure	1.0-2.0
Shear column	Columns whose shear failure precede flexural yielding, except for extremely brittle columns	1.0
Flexural column	Columns whose flexural yielding precede shear failure	1.27-3.2
Extremely brittle column	Columns whose *ho/D* are equal to or smaller than 2 and shear failure precede flexural yielding	0.8

Dominant Members in Second Level Screening

Ductility-dominant basic seismic index of structure

- Vertical members shall be classified by their ductility indices *F* into **three groups or less**
- The index *F* of the first group shall be taken as **larger than 1.0** and the index *F* of the third group shall be less than the ductility index corresponding to the ultimate deformation of the story
- The **minimum ductility index** of the vertical members should be used in **each group**.

Any grouping of members may be adopted so that the index *E0 would be evaluated as maximum*

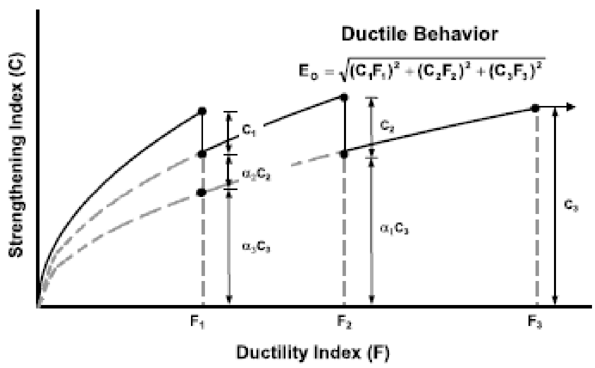
Dominant Members in Second Level Screening (Contd.)

$$E_0 = (n+1)/(n+i)\sqrt{(E_1^2 + E_2^2 + E_3^2)}$$

Where

$E_1 = C_1 F_1$
$E_2 = C_2 F_2$
$E_3 = C_1 F_3$

C_1 = The strength index *C* of the first group (with small *F* index).
C_2 = The strength index *C* of the second group (with medium *F* index).
C_3 = The strength index *C* of the third group (with large *F* index).
F_1 = The ductility index *F* of the first group.
F_2 = The ductility index *F* of the second group.
F_3 = The ductility index *F* of the third group.



Ductility dominant members

Dominant Members in Second Level Screening (Contd.)

Strength-dominant basic seismic index of structure

- **ductility index of the first group *F1*** shall be selected as the cumulative point of strength.
- **contribution of strength** indices of only the vertical members with larger ductility indices than that of the first group shall be considered

Any grouping of members may be adopted so that the index E0 would be evaluated as maximum

$$E_0 = (n+1)/(n+i)\,(C_1 + \Sigma\alpha_j C_j)F_1$$

α_j= Effective Strength Factor in the j-th group at the ultimate deformation R_1 corresponding to the first group (Ductility Index F_1)

Dominant Members in Second Level Screening (Contd.)

Effective Strength Factor

Cumulative point of the first group F_1 =0.8 (Drift angle R_1=R_{500}=1/500)		
	F_1	F_1=0.8
	R_1	R_1=R_{500}
Second and higher groups	Shear (R_{su}=R_{250})	α_S
	Shear (R_{250}< R_{su})	α_S
	Flexural (R_{my}=R_{250})	0.65
	Flexural (R_{250}<R_{my}<R_{150})	α_m
	Flexural (R_{my}=R_{150})	0.51
	Flexural and shear walls	0.65

Dominant Members in Second Level Screening (Contd.)

Effective Strength Factor (Contd.)

Cumulative point of the first group $F_1 \geq 1.0$ (Drift angle $R_1 \geq R_{250}$=1/250)				
	F_1	F_1=1.0	$1.0<F_1<1.27$	$1.27 \leq F_1$
	R_1	R_{250}	$R_{250}<R_1<R_{150}$	$R_{150} \leq R_1$
Second and higher groups	Shear ($R_{su}=R_{250}$)	1.0	0.0	0.0
	Shear ($R_1<R_{su}$)	α_S	α_S	0.0
	Flexural ($R_{my}<R_1$)	1.0	1.0	1.0
	Flexural ($R_1<R_{my}$)	α_m	α_m	1.0
	Flexural ($R_{my}=R_{150}$)	0.72	α_m	1.0

Dominant Members in Second Level Screening (Contd.)

α_S = Effective strength factor of a shear column, calculated by
$\alpha_S = Q_{(F1)}/Q_{su} = \alpha_m\ Q_{mu}/Q_{su} \leq 1.0$

α_m = Effective strength factor of a flexural column, calculated by
$\alpha_m = Q_{(F1)}/Q_{mu} = 0.3+0.7 \times R_1/R_{my}$

R_{my} = Drift angle at flexural yielding, calculated by Eq. (A1.3-1) in the Supplementary Provisions 1.

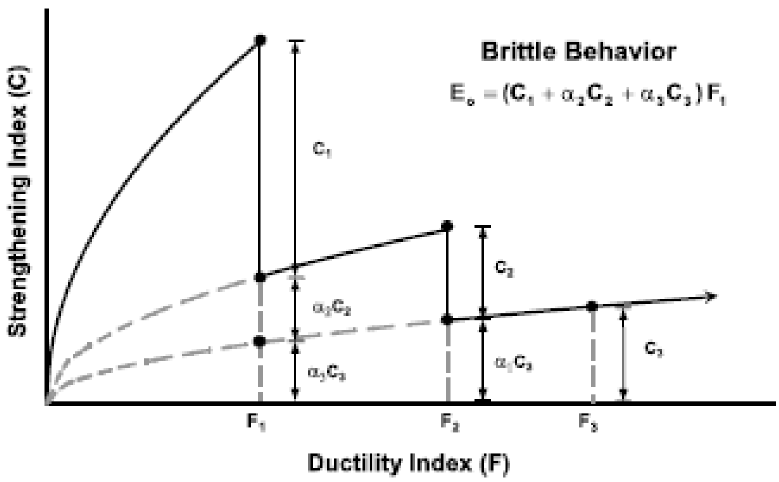
R_{su} = Drift angle at shear strength, calculated by Eq. (A1.2-11) in the Supplementary Provisions 1.

$Q_{(F1)}$ = Shear force at the deformation capacity R_1 of a column in the second and higher groups.

Q_{su} = Shear strength of a column in the second and higher groups (3.2.2).

Q_{mu} = Shear force at flexural yielding of a column in the second and higher groups (3.2.2).

Dominant Members in Second Level Screening (Contd.)



Brittle dominant members

Seismic Index Is after rehabilitation

- If the structural seismic capacity I_S is more than required seismic capacity I_{SO} , the structure is judged safe against earthquake motion observed in **1968 Tokachi Oki earthquake, 1978 Miyagi Ken Oki earthquake or the 1995 Hyogo ken Nanbu** earthquake.
- If Seismic Index I_S is less than index I_{SO} but more than **0.65 I_{SO}** the structure is thought to possess reasonable seismic resistance, but the vulnerability assessment by the second level screening is recommended.

Conclusion

Seismic evaluation technique developed in Standard **for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001** is basically based on the existing RCC buildings of Japan, so some parameters may be modified for using it in any other country.

References

- Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001, Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001 and Technical Manual for Seismic Evaluation and Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001
- Lecture Notes of IISE, BRI for Earthquake Engineering Course by Shunsuke Sugano, Professor Emeritus, Hiroshima University, Visiting Research Fellow, IISEE, BRI.
- Shunsuke OTANI, Professor, University of Tokyo, "Seismic Vulnerability of Reinforced Concrete Building."
- Toshimi Kabeyasawa, Professor, University of Tokyo, "Improvement of Seismic Performance of Reinforced Concrete School Building in Japan Part1 Damage Survey and Performance Evaluation after 1995 Hyogo- Ken Nambu Earthquake.


Thank You Very Much

SHORT TRAINING COURSE ON SEISMIC ASSESSMENT, RETROFIT DESIGN AND CONSTRUCTION OF RC BUILDING

TITLE OF LECTURE

CONCEPT ON RETROFITTING DESIGN

PRESENTED BY

ANUP KUMAR HALDER

SUB DIVISIONAL ENGINEER

PWDDESIGN DIVISION-V.

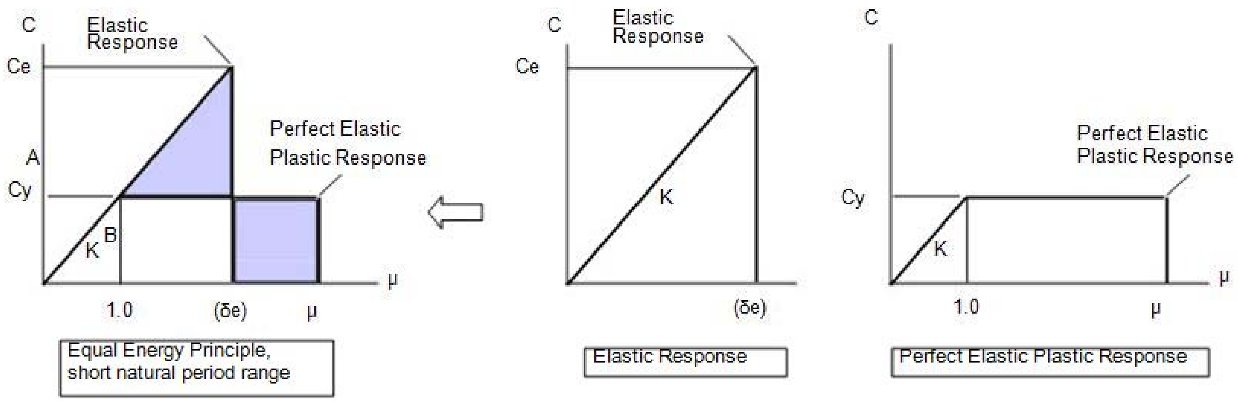
&

TEAM MEMBER WORKING TEAM-II

OUTLINE

1. BASIC CONCEPT
2. SIGNIFICANCE OF “C”
3. SIGNIFICANCE OF “F”
4. SHEAR COLUMN
5. FLEXURAL COLUMN
6. STRENGTH DOMINANT & DUCTILITY DOMINANT STRUCTURE
7. SIGNIFICANCE OF “E_O”
8. ESTABLISHMENT OF“I_{SO}” VALUE
9. IMPORTANCE OF “S_D”
10. IMPORTANCE OF “T”
11. JUDGEMENT OF “I_{SO}” VALUE
12. SEISMIC PERFORMANCE LEVEL AS PER ASCE-41
13. ANALYSIS & ACCEPTANCE CRITERION ASCE-41
14. RETROFITTING METHODS
15. STRENGTHENING EFFECT OBSERVED
16. STRATEGIES & PLANNING

BASIC CONCEPT



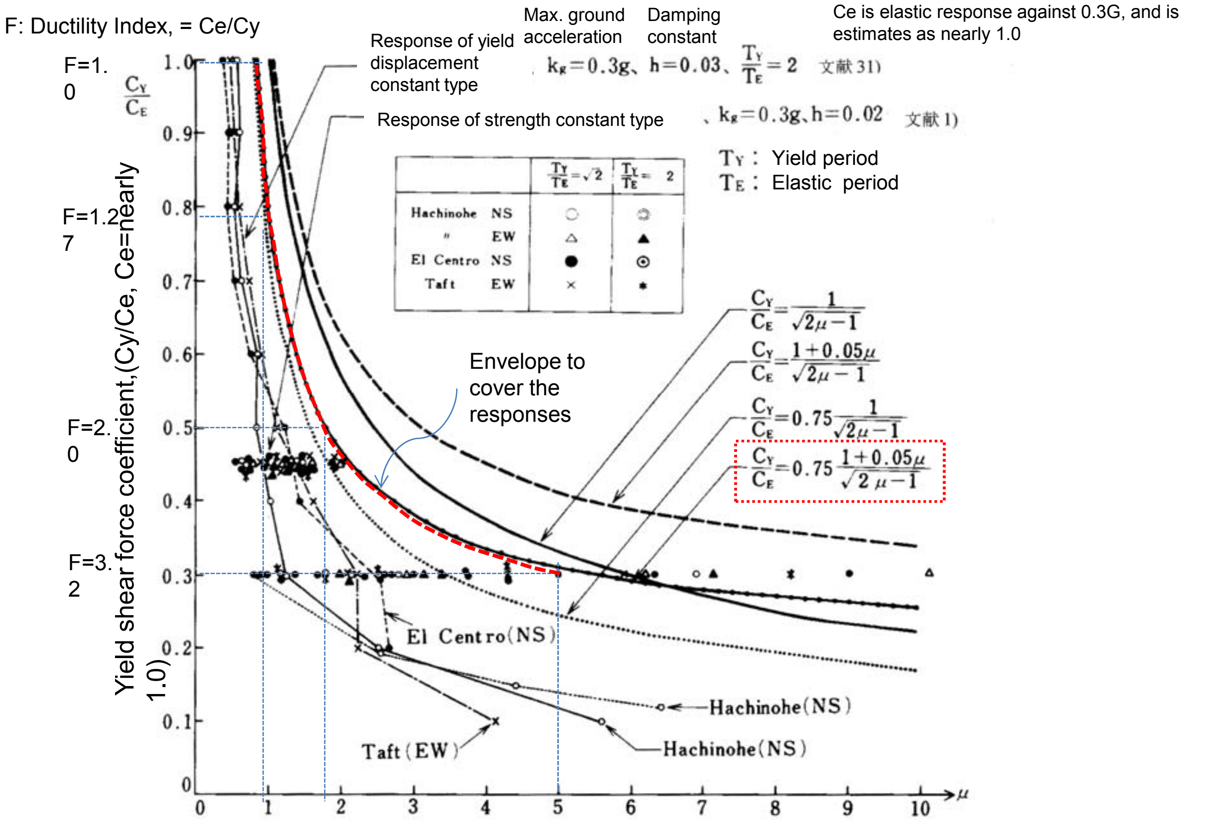
ELASTIC RESPONSE	ELASTIC PLASTIC RESPONSE
GROUND MOTION PRODUCES ELASTIC RESPONSE BASE SHEAR (Ce)	MIN BASE SHEAR COEFFICIENT FOR STUCTURE SYSTEM (Cy)
	DUCTILITY μ (DEFORMATION CAPACITY/YIELD DEFORMATION.)

$C_e = C_y\sqrt{2\mu - 1}$ for short period systems

$C_e = C_y \cdot \mu$ for long period systems

$E_0 = C \cdot F$ FOR A SINGLE DEGREE OF FREEDOM SYSTEM STRUCTURAL RESISTANCE

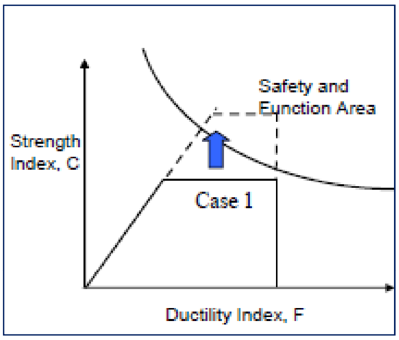
Response ductility factor “μ” and Yield shear force coefficient “Cy/Ce” or ductility index “F (= Ce/Cy)” by Degrading Tri-linear RC frame model



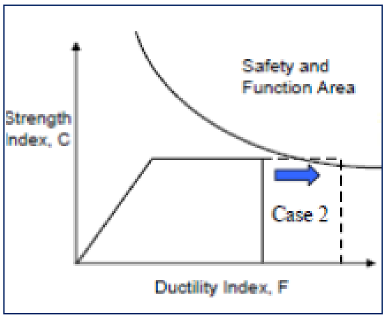
Source; Japan Building Disaster Prevention Association, "Seismic evaluation of existing RC buildings 2001"(Japanese

BASIC CONCEPT cont…

1. INCREASING STRENGTH

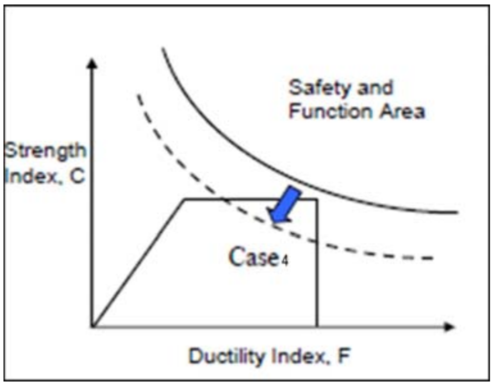


2. INCREASING DUCTILITY



3. INCREASING STRENGTH+ DUCTILITY

4. REDUCTION OF SEISMIC LOAD



5. IMPROVE CONFIGURATION

SIGNIFICANCE OF "C"

- LATERAL STRENGTH OR LOAD CARRYING CAPACITY OF A MEMBER

$$C_C = \frac{\tau_C \cdot A_C}{\Sigma W} \cdot \beta_c \qquad \beta_c = \frac{F_c}{20} \quad F_c \le 20 \qquad \tau_C = 1\ \text{N/mm}^2$$

$$\beta_c = \sqrt{\frac{F_c}{20}} \quad F_c > 20$$

1ST LEVEL

Story	∑W	T. Ac (mm²)	f'c(Mpa)	(n+1)/(n+i)	β_c	Cc
5	3021	5625000	17	0.60	0.85	1.58
4	7703	5625000	17	0.67	0.85	0.62
3	12386	5625000	17	0.75	0.85	0.39
2	17068	5625000	17	0.86	0.85	0.28
1	21750	5625000	17	1.00	0.85	0.22

$$C = \frac{Q_u}{\sum W}$$

Frame	FL	∑W (KN)	Qu	C
	5	**525**	199	0.3795
	4	**1050**	211	0.2013
	3	**1557**	223	0.1435
	2	**2082**	235	0.1131
2A	1	**2624**	248	0.0943

2ND LEVEL

SIGNIFICANCE OF "F"

- DEFORMATION CAPACITY OF STRUCTURAL MEMBER

Vertical member	Ductility index F
Column ($h_0/D>2$)	1.0
Extremely short column ($h_0/D\leq 2$)	0.8
Wall	1.0

1ST LEVEL

- SHEAR COLUMN

$$F = 1.0 + 0.27\frac{R_{su} - R_{250}}{R_y - R_{250}}$$

- FLEXURAL COLUMN

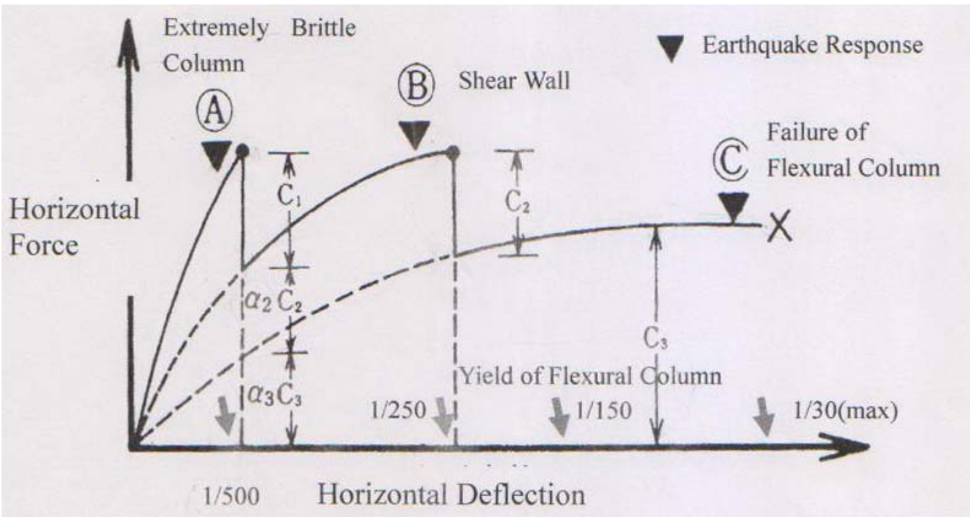
2ND LEVEL

$$F = 1.0 + 0.27\frac{R_{mu} - R_{250}}{R_y - R_{250}} \qquad R_{mu} < R_y$$

$$F = \frac{\sqrt{2R_{mu}/R_y - 1}}{0.75\cdot(1 + 0.05R_{mu}/R_y)} \leq 3.2 \qquad R_{mu} \geq R_y$$

SIGNIFICANCE OF "F" cont…..

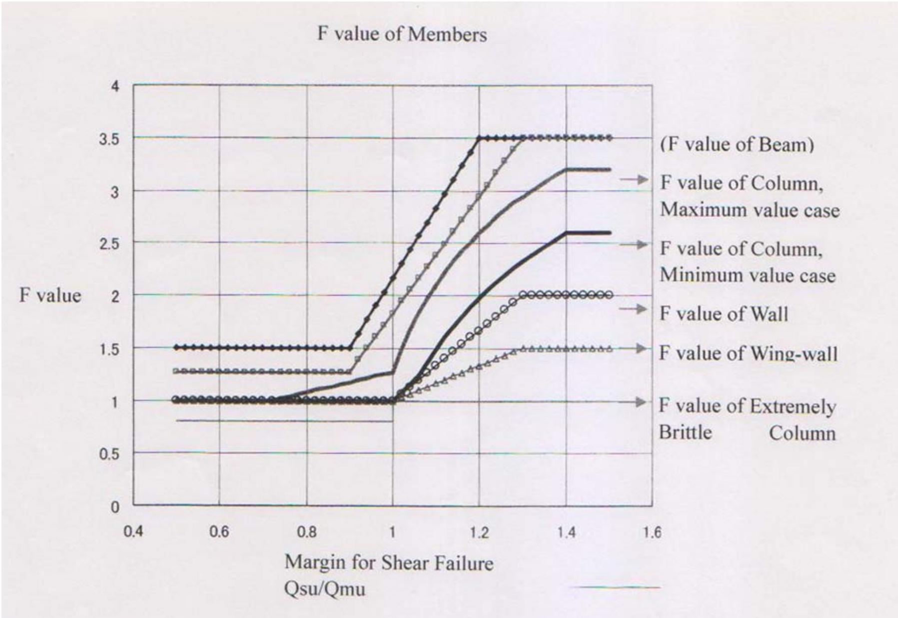
(STANDARD DEFORMATION ANGLE)



Following conditions are assumed in this Code;

Yield deflection storey angle for standard column:	1/150	=0.0066
Ultimate storey deflection angle of shear wall:	1/250	=0.0040
Ultimate storey deflection angle of extremely brittle columns:	1/500	=0.0020

SIGNIFICANCE OF “F” cont…. (Margin of shear failure)



3RD LEVEL

SIGNIFICANCE OF “F” cont…. (Based on Ductility ratio)

DUCTILITY CAPACITY OF A FLEXURAL COLUMN :

$$1 \le \mu = \mu_o - k_1 - k_2 \le 5$$

$$\mu_o = 10\left(\frac{{}_C Q_{su}}{{}_C Q_{mu}} - 1\right)$$

$$k_1 = 2.0$$

(K1=1; WHEN HOOP SPACING 8TIMES THE DIA OF MAIN RE BAR)

$$k_2 = 30\left(\frac{{}_C \tau_{mu}}{F_c} - 1\right) \ge 0$$

$${}_C \tau_{mu} = {}_C Q_{mu} / (b \cdot j)$$

DUCTILITY INDEX F=1; IF FOLLOWING CONDITION IS SATISFIED

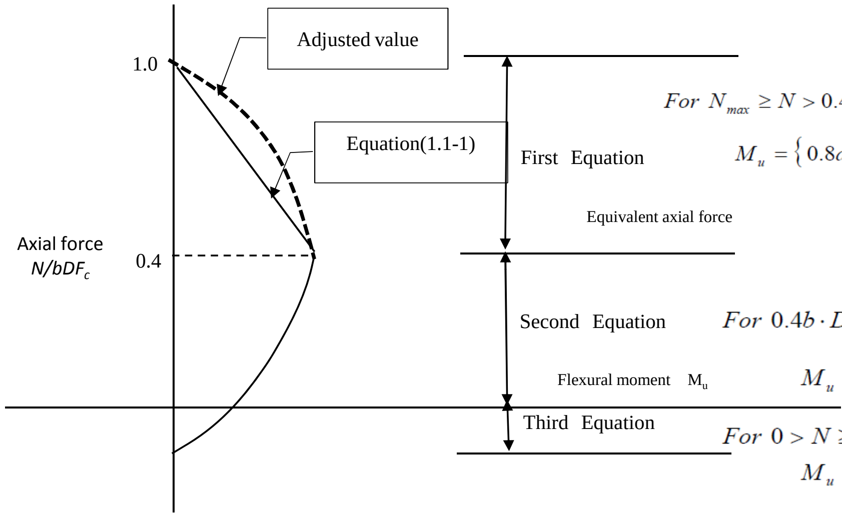
$$N_s / (bDF_c) > 0.4$$

$${}_C \tau_{mn} / F_c > 0.2$$

$$P_t > 1\%$$

$$h_o / D \le 2.0$$

FLEXURAL COLUMN



$For\ N_{max} \geq N > 0.4b \cdot D \cdot F_c$

$$M_u = \left\{0.8a_t \cdot \sigma_y \cdot D + 0.12b \cdot D^2 \cdot F_c\right\} \cdot \left(\frac{N_{max} - N}{N_{max} - 0.4b \cdot D \cdot F_c}\right)$$

$For\ 0.4b \cdot D \cdot F_c \geq N > 0$

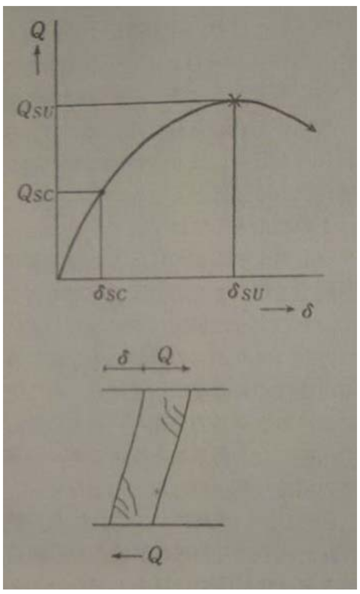
$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.5N \cdot D \cdot \left(1 - \frac{N}{b \cdot D \cdot F_c}\right)$$

$For\ 0 > N \geq N_{min}$

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.4N \cdot D$$

$$Q_{mu} = 2 \cdot M_u / h_0$$

SHEAR COLUMN



FROM EMPIRICAL EQUATION

MAIN REBAR RATIO ,
CONCRETE STRENGTH

AXIAL FORCE RATIO

$$Q_{su} = \left\{\frac{0.053 p_t^{0.23}(18 + F_c)}{M/(Q \cdot d) + 0.12} + 0.85\sqrt{p_w \cdot {}_s\sigma_{wy}} + 0.1\sigma_0\right\} \cdot b \cdot j$$

SLENDERNESS

SHEAR
REINFORCEMENT

STRENGTH TYPE & DUCTILITY TYPE STRUCTURE

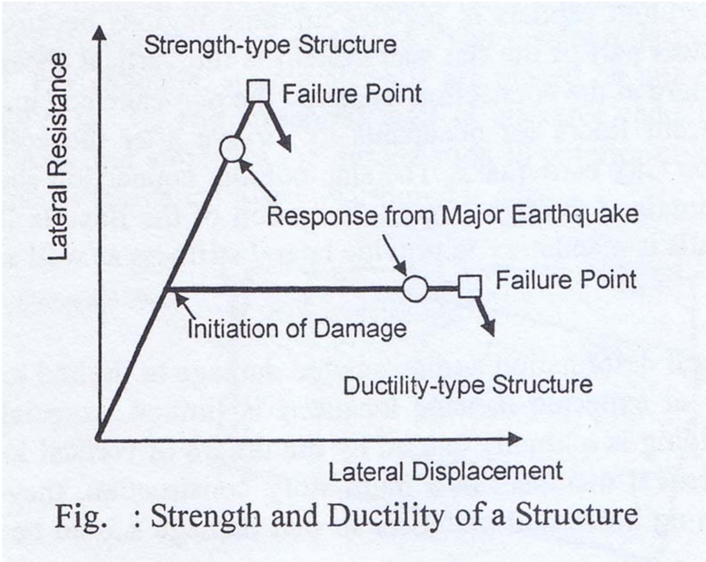
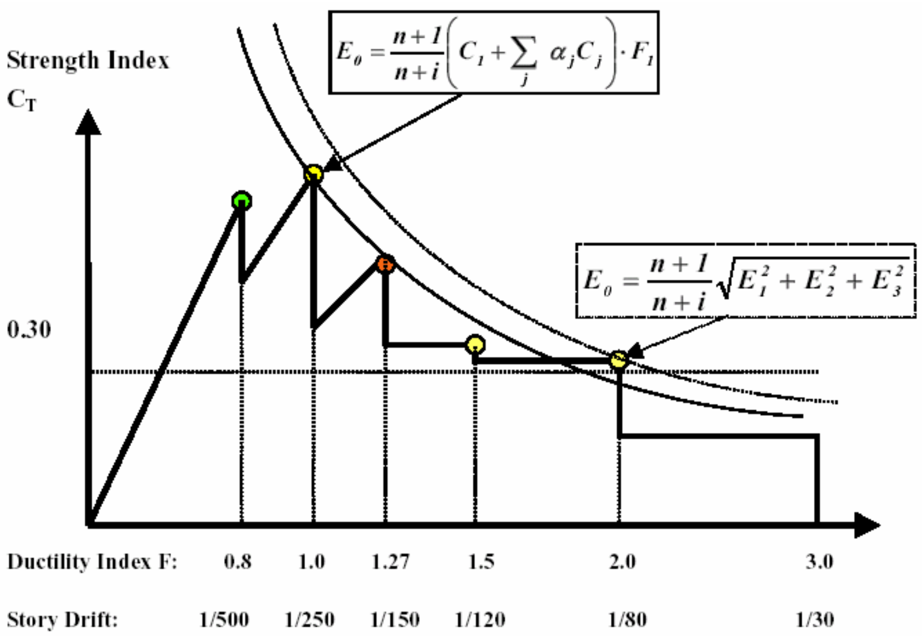


Fig. : Strength and Ductility of a Structure

SOURCE: PROFESSOR SHUNSUKE OTANI'S PAPER

SIGNIFICANCE OF E_0



Idealized relations of lateral strength and ductility for seismic index

SOURCE: PROFESSOR KABAYASAWA'S PAPER

ESTABLISHMENT OF **Iso** (based on 1st level)

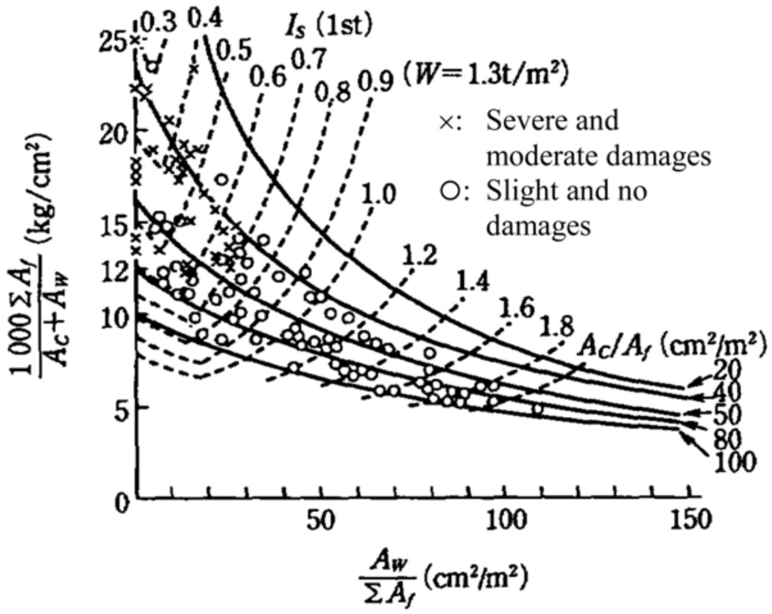
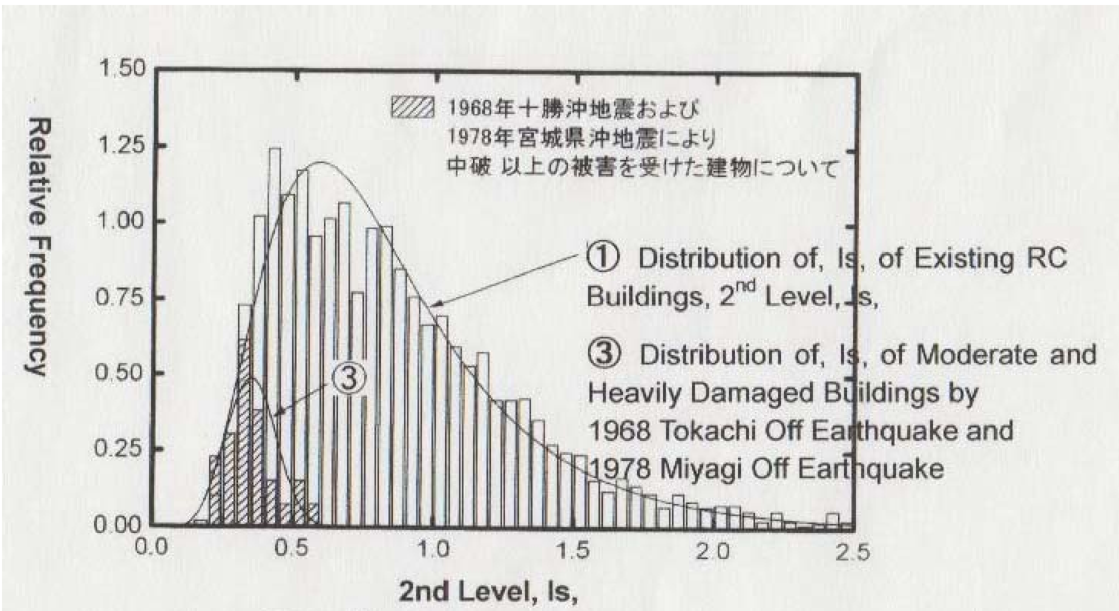


Figure TN.1-3 Index I_S and building damage
(1968 Tokachi-oki and 1978 Miyagiken-oki earthquakes)
(quoted from Figure 4 on page 511 of Ref. 1)

ESTABLISHMENT OF **Iso** (based on 2nd level)

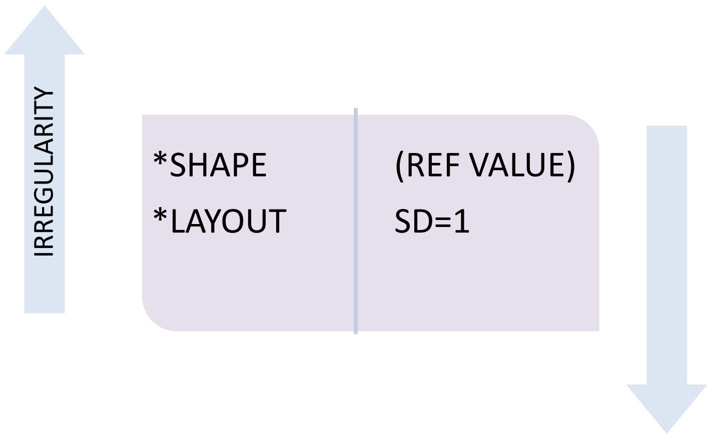


(Figure A5.2-3 of the Japanese Code)

IMPORTANCE OF **S**D

IT MODIFY SEISMIC INDEX BY QUANTIFYING THE EFFECT OF

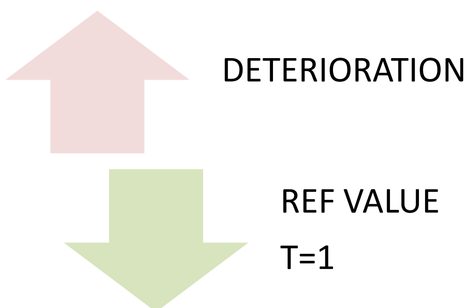
- HORIZONTAL BALANCE
- ELEVATION BALANCE
- ECCENTRICITY
- STIFFNESS



IMPORTANCE OF **T**

TIME INDEX EVALUATES THE EFFECTS OF STRUCURAL DEFECTS

- STRUCTURAL CRACKING AND DEFLECTION
- DETERIORATION AND AGING.



JUDGEMENT

$$I_S \geq I_{SO}$$

where:

I_S = Seismic index of structure

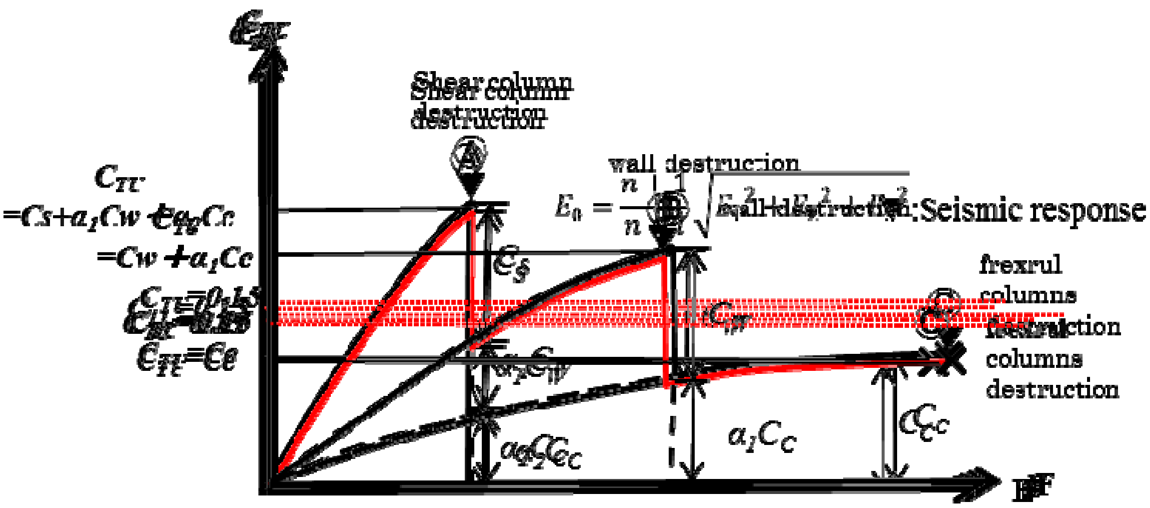
I_{SO} = Seismic demand index of structure

$$C_{TU} \cdot S_D \geq 0.3 \cdot Z \cdot G \cdot U$$

C_{TU} = Cumulative strength index at the ultimate deformation of structure.

S_D = Irregurality index.

JUDGEMENT



JUDGEMENT

$$I_{SO} = E_S \cdot Z \cdot G \cdot U$$

E_S = Basic seismic demand index of structure, standard values of which shall be selected as follows regardless of the direction of the building:

$E_S = 0.8$ for the first level screening,

$E_S = 0.6$ for the second level screening, and

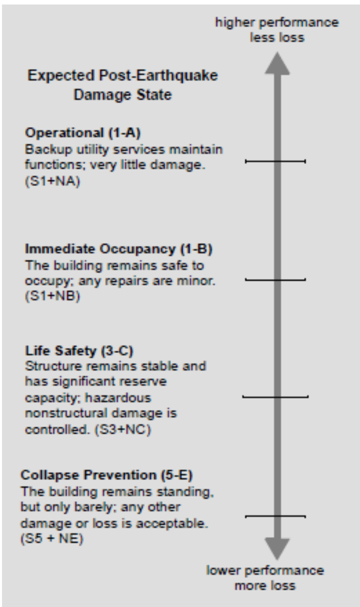
$E_S = 0.6$ for the third level screening.

Z = Zone index, namely the modification factor accounting for the seismic activities and the seismic intensities expected in the region of the site.

G = Ground index, namely the modification factor accounting for the effects of the amplification of the surface soil, geological conditions and soil-and-structure interaction on the expected earthquake motions.

U = Usage index, namely the modification factor accounting for the use of the building.

SEISMIC PERFORMANCE LEVELS



TARGET BUILDINGS PERFORMANCE LEVEL (ASCE-41)

PERFORMANCE LEVEL ASCE 41

Table C1-2. Damage Control and Building Performance Levels

Table C1-3. Structural Performance Levels and Damage[1,2,3]—Vertical Elements

Elements	Type	Structural Performance Levels: Collapse Prevention (S-5)	Life Safety (S-3)	Immediate Occupancy (S-1)
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (< 1/8-in. width) for ductile columns. Minor spalling in nonductile columns. Joint cracks < 1/8 in. wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints < 1/16-in. width.
	Drift	4% transient or permanent.	2% transient; 1% permanent.	1% transient; negligible permanent.

Table C1-4. Structural Performance Levels and Damage[1,2]—Horizontal Elements

Table C1-6. Nonstructural Performance Levels and Damage[1]—Mechanical, Electrical, and Plumbing Systems/Components

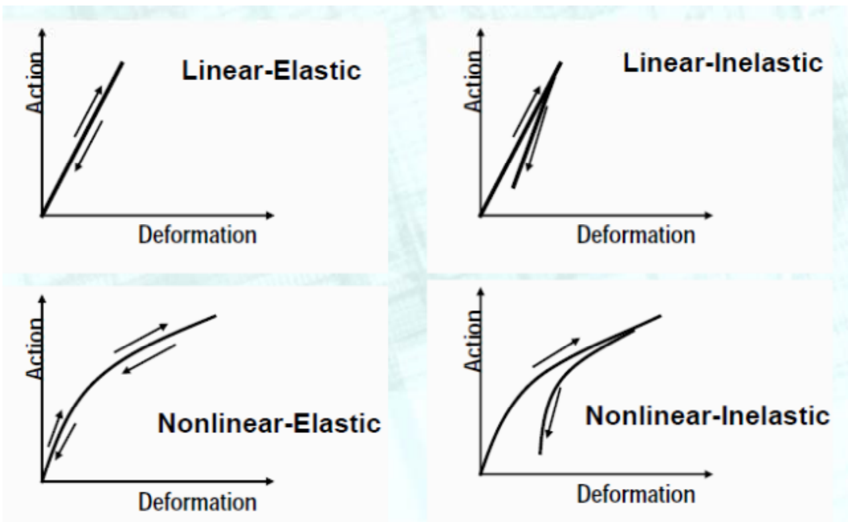
Table C1-5. Nonstructural Performance Levels and Damage[1]—Architectural Components

Table C1-7. Nonstructural Performance Levels and Damage[1]—Contents

Table C1-8. Target Building Performance Levels and Ranges

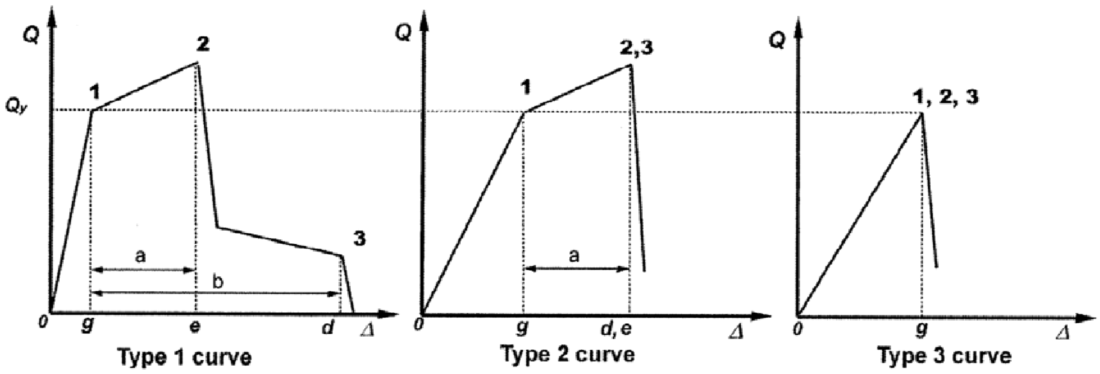
ANALYSIS PROCEDURE (ASCE-41)

- 1. LINEAR STATIC
- 2. LINEAR-DYNAMIC
- 3. NONLINEAR STATIC
- 4. NONLINEAR-DYNAMIC



ACCEPTANCE CRITERIA (ASCE-41)

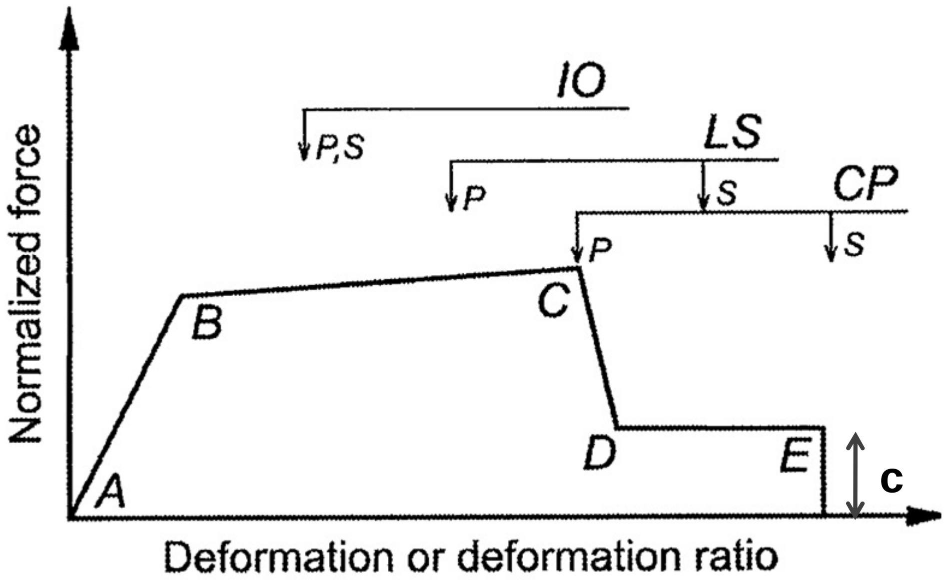
- PRIMARY COMPONENT (P)
- SECONDARY COMPONENT (S)
- DEFORMATION CONTROLLED ACTION
- FORCE CONTROLLED ACTION



DEFORMATION CONTROLLED (e≥2g)

Component Force Versus Deformation Curves.

ACCEPTANCE CRITERIA cont….(ASCE-41)



COMPONENT OR ELEMENT DEFORMATION ACCEPTANCE CRITERIA

NUMERICAL ACCEPTANCE CRITERIA FOR COLUMNS, ASCE-41

Table 6-8 *Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Columns*

Conditions			Modeling Parameters[4] Plastic Rotation Angle, radians		Residual Strength Ratio	Acceptance Criteria[4] Plastic Rotation Angle, radians — Performance Level				
							Component Type: Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear[1, 3]										
All cases[5]			—	—	—	—	—	—	.0030	.0040
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ d/2			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing > d/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$[1, 3]										
Conforming hoops over the entire length			0.015	0.025	0.02	0.0	0.005	0.01	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

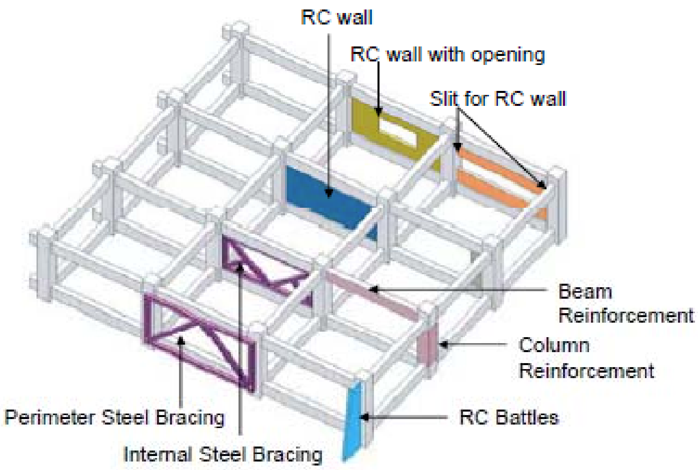
0.0066 (1/150)

0.0040 (1/250)

RETROFITTING METHODS

A. STRENGTH UPGRADING	1. ADDING WALL 2. STEEL WITH FRAME 3. EXTERIOR STEEL FRAME 4. STRUCTURAL FRAME 5. OTHERS
B. DUCTILITY UPGRADING	1. RC JACKETING 2. STEEL JACKETING 3. FRP WRAPING
C. PREVENTION OF DAMAGE CONNECTION	1. IMPROVEMENT OF VIBRATION PROPERTY 2. IMPROVEMENT OF EXTREME BRITTLE MEMBER
D. REDUCTION OF SEISMIC FORCES	1. MASS REDUCTION 2. SEISMIC ISOLATION 3. STRUCTURAL RESPONSE DEVICE
E. STRENGTHENING OF FOUNDATION	1. STRENGTHENING FOUNDATION BEAM 2. STRENGTHENING OF PILE

RETROFITTING METHODS cont..



Building Contractors Society (BCS), Japan
'Seismic Retrofitting Brochure 2006'

STRENGTHENING EFFECT OBSERVED IN STRUCTURAL TEST

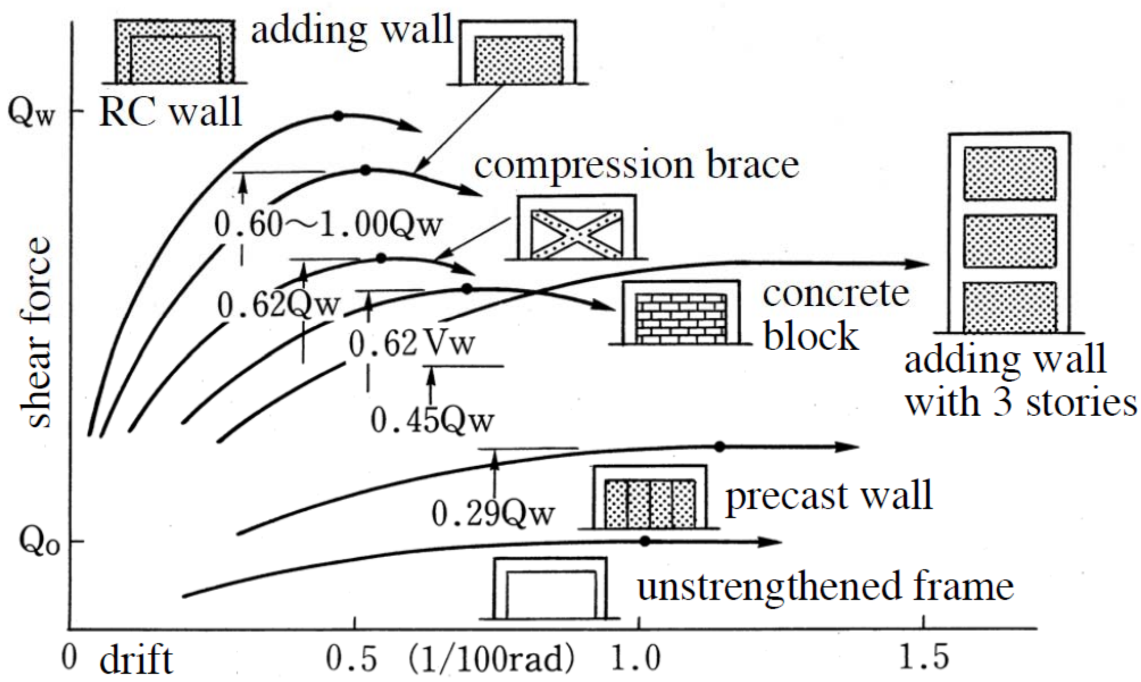
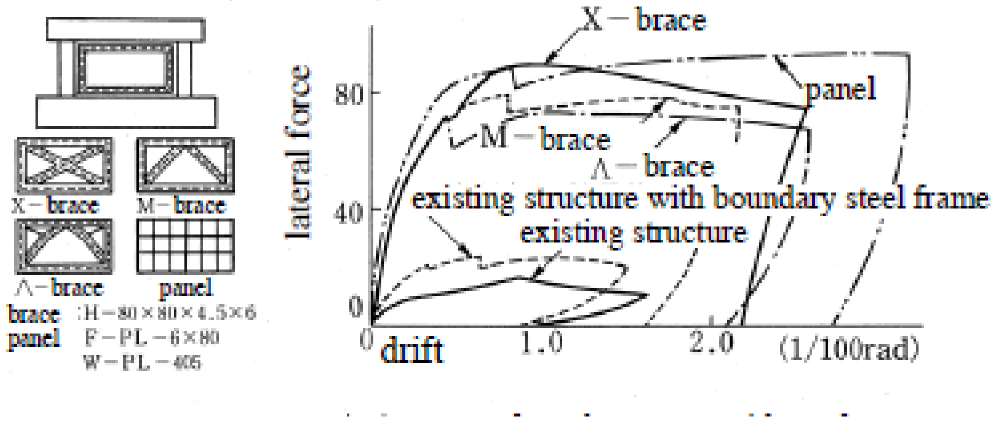


Figure TN.9 Strengthening effect observed in previous structural tests

(quoted from the figure on page 73 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)

(a) strengthening of frame

STRENGTHENING EFFECT OBSERVED IN STRUCTURAL TEST cont..

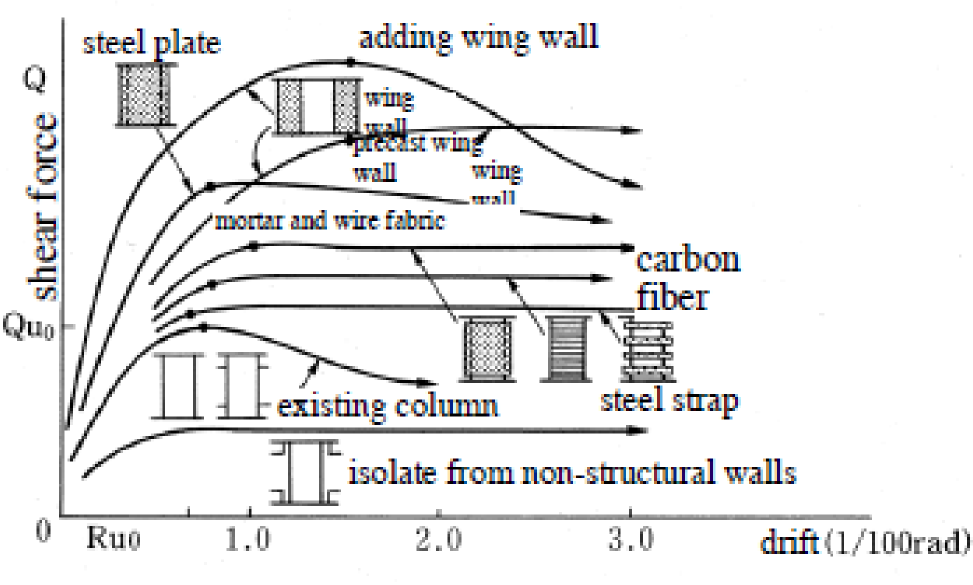


(b) strengthened structure with steel brace with boundary steel frame

Figure TN.9 Strengthening effect observed in previous structural tests

(quoted from the figure on page 73 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)

STRENGTHENING EFFECT OBSERVED IN STRUCTURAL TEST cont....



(c) strengthening of column

Figure TN.9 Strengthening effect observed in previous structural tests

(quoted from the figure on page 73 in the commentary of 2.1.2 of the Guidelines of 2001 Japanese version)

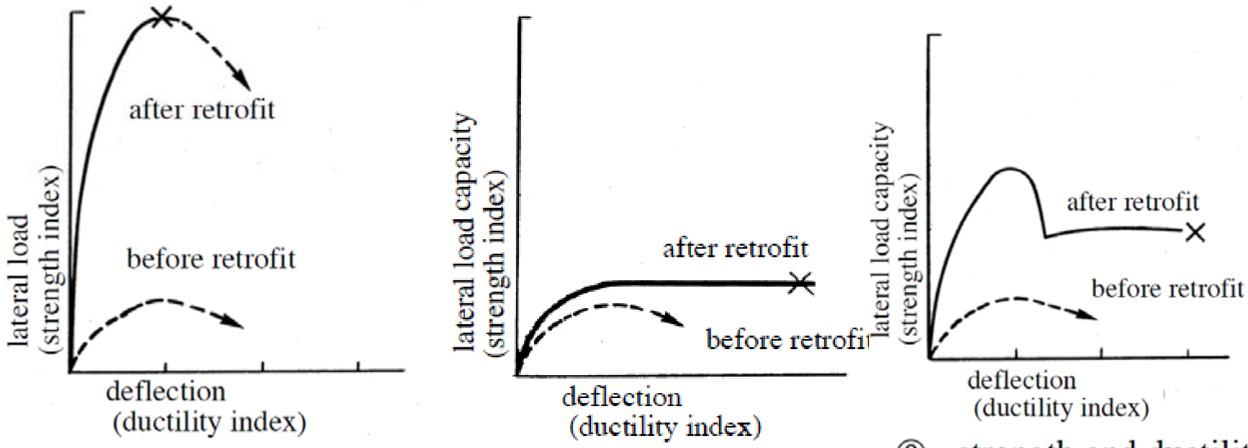
STRATEGIES

- IMPROVING REGULARITIES
- STRENGTHENING
- DUCTILITY
- DAMPING
- MASS REDUCTION
- CHANGING USE

DESIGN PROCEDURE

- PLANNING
- STRUCTURAL DESIGN
- DETAILED DESIGN
- EVALUATION OF RETROFIT EFFECT

PLANNING & STRUCTURAL DESIGN cont..



① strength upgrading ② ductility upgrading ③ strength and ductility

Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

RETROFITTING DESIGN METHODS

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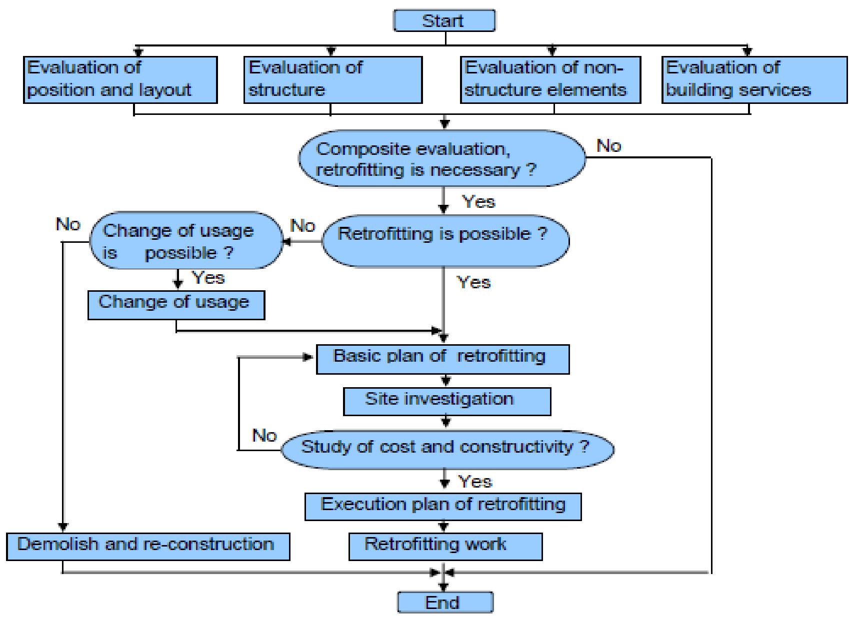
PUBLIC WORKS DEPARTMENT AND
TEAM MEMBER, COMPONENT-2,
CNCRP PROJECT

A flow chart of Seismic Evaluation and Retrofitting for public buildings (facilities), Japan

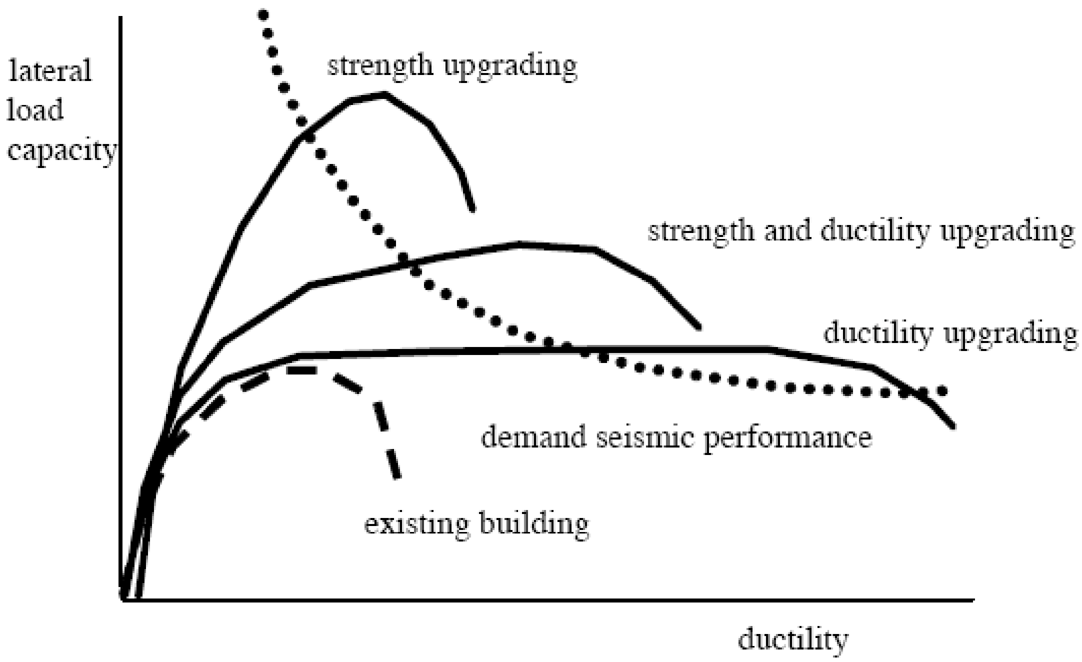


Source: Building Integrity Center, 1996 "Guideline and explanation of composite seismic evaluation and retrofitting for public facilities (in Japanese)"

Methods of Retrofitting

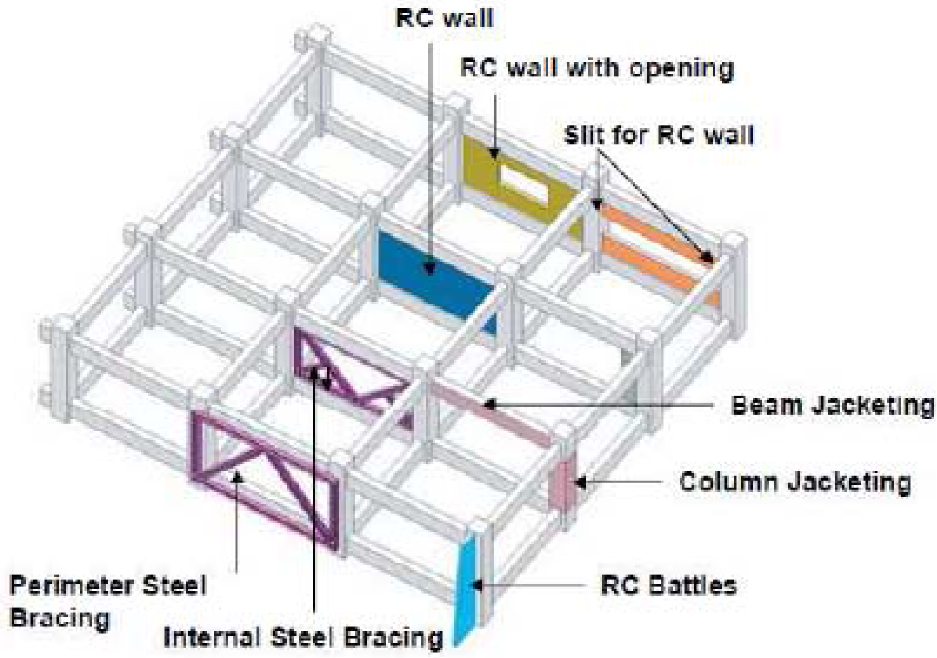
Types of Retrofitting Methods				
No.	Description of Retrofitting Methods	Improvement of Strength	Improvement of Ductility	Improvement of Structural Balance
1	Steel Framed Bracing	○		
2	Infilling New RC Shear Wall into Open Frame	○		○
3	Increasing Thickness of Existing Shear Wall	○		○
4	Infilling Steel Plate Wall into Open Frame	○		○
5	Constructing New RC Wing Wall to RC Column	○		
6	Constructing External Frame	○		
7	Constructing External Buttress	○		
8	Steel Plate Jacketing around RC Column		○	
9	Carbon Fiber (Sheet / Strand) Wrapping around RC Column		○	
10	Concrete Jacketing around RC Column	○	○	
11	Providing New Seismic Slit		○	○

Seismic Index of Structure (Is)



Source: Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001 (English version, 1st edition),

Methods of Retrofitting



Source: Building Contractors Society (BCS), Japan 'Seismic Retrofitting Brochure 2006'

Basics of Retrofitting Design

Seismic Index of Structure,

$Is = EoS_DT$ (1)

whrere: **Eo** : Basic Seismic Index of Structure

S_D : Irregularity Index

T : Time Index

Eo as the larger one from eqs (4) and (5). Each equation is calculated within the limitation of the maximum ductility index.

Eo of ductility-dominant Structure,

$Eo = (n+1/n+i)*\sqrt{(C1*F1)^2+(C2*F2)^2+(C3*F3)^2}$ (4)

Eo of strength-dominant Structure,

$Eo = (n+1/n+i)*(C1+\Sigma\alpha j\ Cj)F1$ (5)

whrere: **C** : Strength Index,

F : Ductility Index, Ductility Index is estimated mainly depending on the margin of members against shear failure.

n+1/n+i : Storey-shear modification factor

α : Effective strength factor

$C = Qu/\Sigma W$ (12)

Qu :Ultimate lateral load-carrying capacity of the vertical members in the storey concerned

ΣW :Total weight supported by the storey concerned

Source: Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001 (English version, 1st edition),

Check no -1:

$I_S \geq I_{so}$

I_{So} = Seismic demand index of structure

$= E_S . Z. G. U$

E_s = Basic seismic demand index of structure

Z = Zone index

G = Ground index

U = Usage index

Check no -2:

$C_{TU}. S_D \geq 0.3 . Z . G . U$

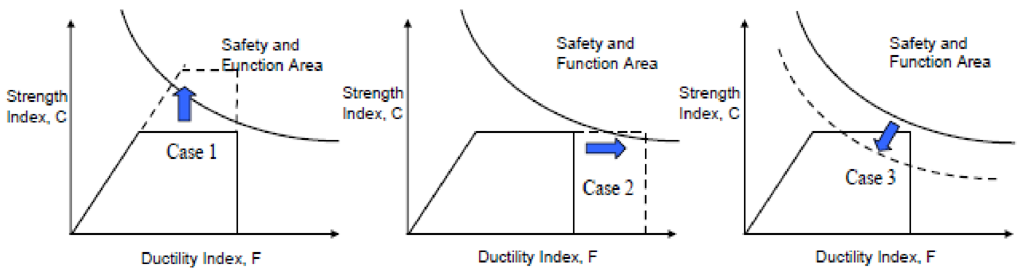
C_{TU} = Cumulative strength index at ultimate deformation of structure

S_D = Irregularity index

Source: Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, 2001 (English version, 1st edition),

Concept of Seismic Retrofitting - Combination of strength and ductility

1 Increasing strength (Case 1 of following figure)
2 Increasing ductility (Case 2 of following figure)
3 Improvement of configuration (Case 3 of following figure)
4 Reduction of seismic load (Case 3 of following figure)

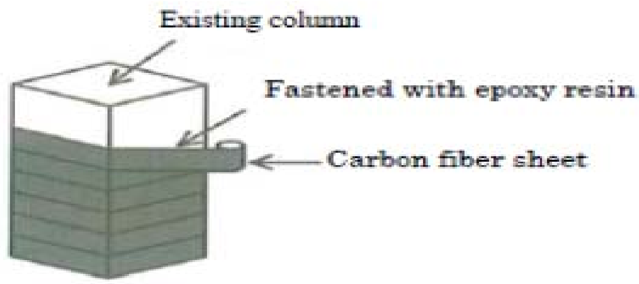


[Source: "Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001", The Japan Building Disaster Prevention Association (English version)]

Outline of retrofitting method

1.Carbon Fiber Sheet Wrapping around RC Column

Existing columns in buildings are wrapped with carbon fibre sheets



STRENGTH OF COLUMN AFTER CARBON FIBER WRAPPING

Shear strength of Column

$$Qsu = [0.053\, P_{t2}^{0.23}(F_{c1}+18)/(M/Q\, d+0.12)$$

$$+0.85\, Sqrt(P_w\sigma_{wy}+P_{wf}\sigma_{fd})+0.1\sigma_o]bj$$

p_{t2}= tensile reinforcement ratio of existing column in %

p_w= shear reinforcement ratio of existing column in decimal

p_{wf}= shear reinforcement ratio of carbon fiber sheet in decimal

F_{c1} =compressive strength of concrete for existing structure, N/mm^2

M/ Qd ranges from 1 to 3 and j= 0.8D

b= width of column and D= depth of column

σ_0=axial compressive stress and maximum value 7.8 N/mm^2

d= effective depth of column

σ_{fd}=tensile strength of carbon fiber sheet for shear design

Retrofitting with Carbon Fiber Wrapping

Main features of Carbon Fiber Wrapping:

•Carbon fiber sheet is wrapped with epoxy resin around existing column.

•This method is done for upgrading ductility.

•Construction shall be done by skilled worker since performance of this method is highly dependent construction quality

•Overlap of carbon fiber sheet shall be long enough to ensure the rupture of the material.

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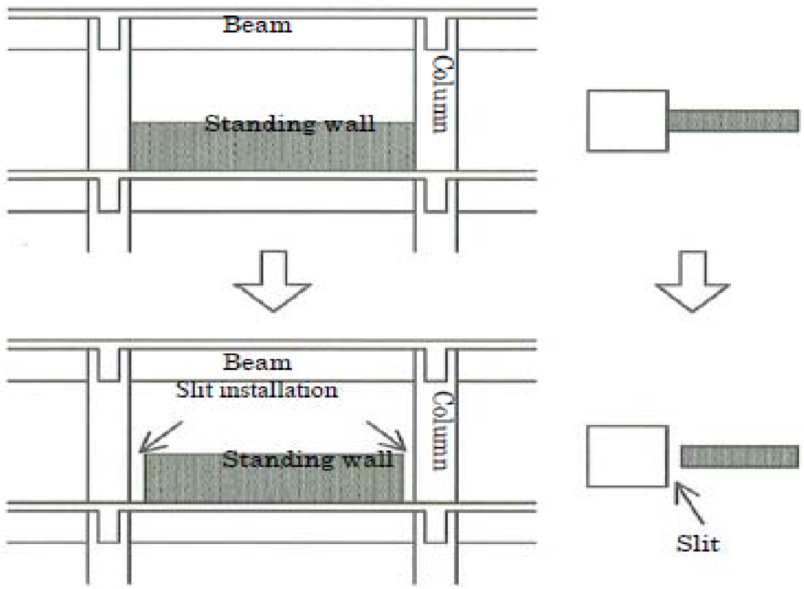
Kensetsu Kaikan Building

2. Providing New Seismic Slit

Slits (open joint) are provided between columns and attached standing walls or wing walls



Retrofitting with Structural Slit

Main features of Structural slit:

•Structural slit may be provided in brick wall or RC wall adjacent to column.

•Improve ductility by avoiding short column.

•Secure safety against out of plane behavior of wall to be cut.

•Secure water proofing performance.

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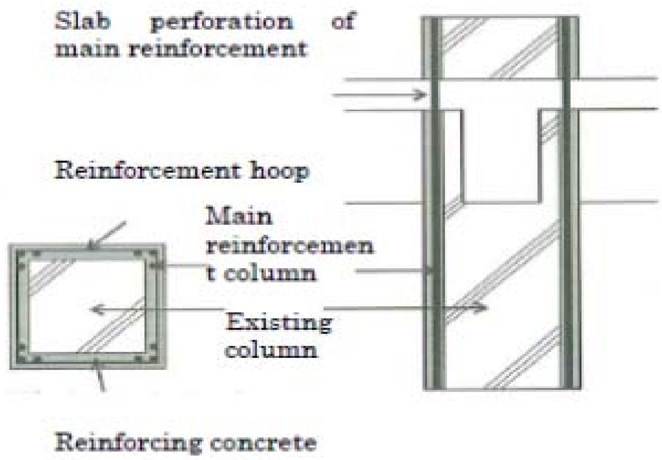
3: Concrete Jacketing around RC Column

Reinforced concrete of a thickness of around 10-15cm is jacketed around

existing building columns



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STRENGTH OF COLUMN AFTER RC JACKETING

Ultimate Flexural Strength of Jacketed Column

$M_u = a_t \sigma_y g + a_{t2} \sigma_{y2} g_2 + 0.5 N D_2[1-N/(b_2 D_2 F_{c1})]$

Shear force by flexural strength

$Q_{mu} = 2M_u/h$

Ultimate Shear Strength of Jacketed Column

$Q_{su} = \varphi [0.053 P_{t2}^{0.23}(F_{c1}+18)/(M/Q\, d_2+0.12)$

$+0.85\, Sqrt(P_w.\sigma_{wy} + P_{w2}.\sigma_{wy2})+0.1\, N/b_2 D_2]0.8\, b_2 D_2$

F_{c1} =compressive strength of concrete for existing structure, N/mm^2

p_{t2}=tensile reinforcement ratio of jacketed column in %

Retrofitting with Column Jacketing

Main features of RC column jacketing:

•Cross section of existing column is increased.

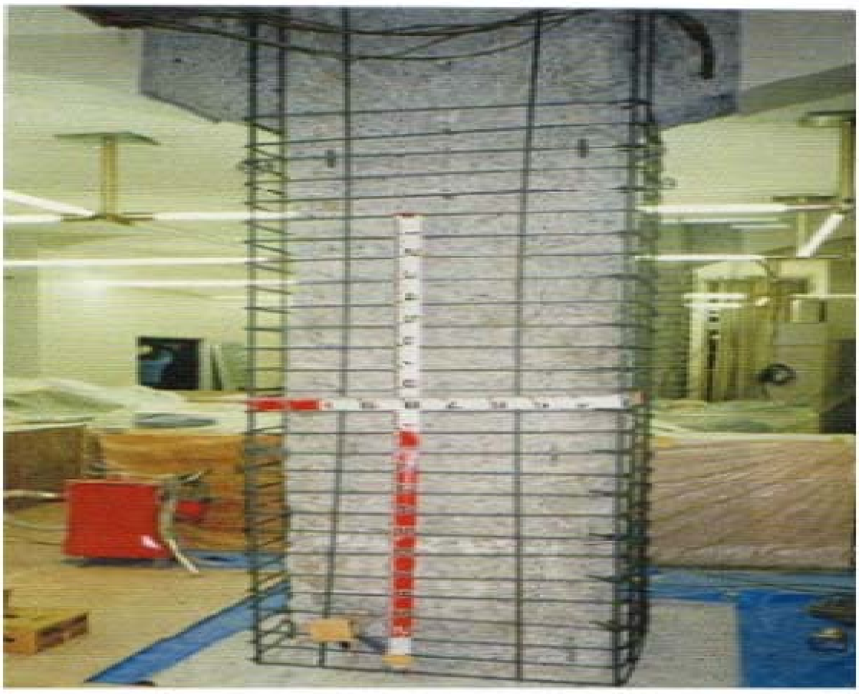
•Usual thickness of jacket is 10 to 15 cm with reinforced concrete.

•Retrofit to improve ductility only.

•Retrofit to improve both ductility and strength.

•In case ductility upgrading provide slit at top and bottom of the column.

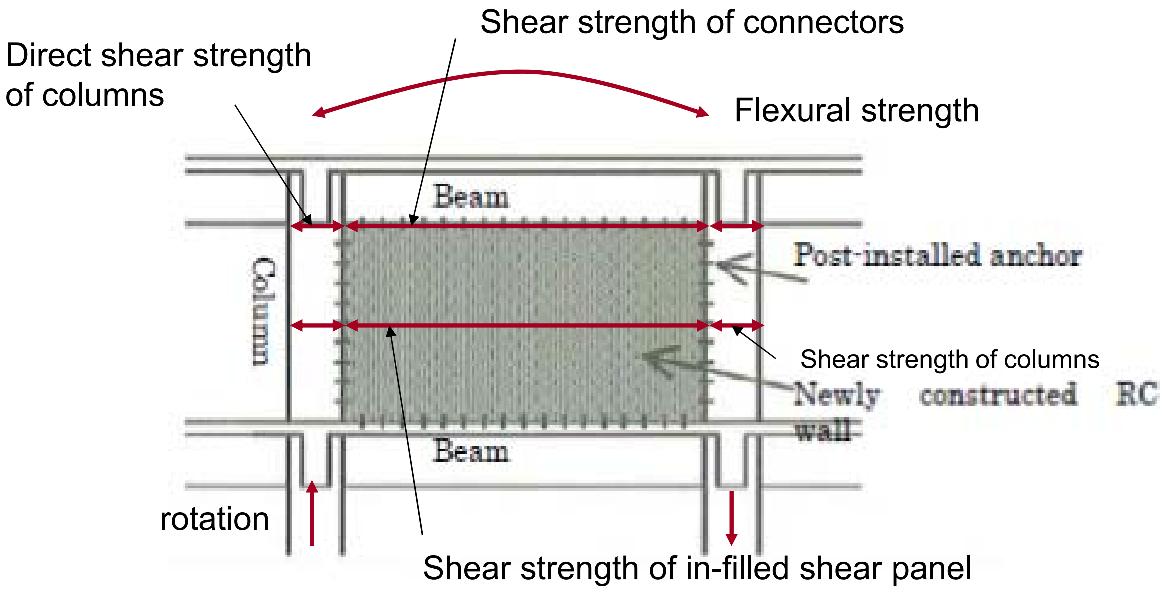
•In case of strength upgrading provide shear key.



Kensetsu Kaikan Building

4: Infilling New RC Shear Wall into Open Frame

Reinforced concrete walls (RC walls) are newly constructed inside existing building column/beam frames



CAPACITY OF INFILLED SHEAR WALL

Shear strength of column:

wQ_{su}=min {wQ'_{su}+ 2∝ $Q_{c,}Q_j$+pQ_c+∝.$Q_{c,}$}

Shear strength of infilled shear panel

wQ'_{su}=max(ρ$_w$,wσ_y,$\frac{F_{cw}}{20}$+0.5ρ$_w$.wσ_y).$t_{w.}l'$

Q_c=Smaller value of the other column between

the shear force at the yielding and shear strength.

p_w, $_w\sigma_y$= wall reinforcement ratio and yield strength of wall bar, N/mm^2

F_{cw}= concrete strength of installed wall panels, N/mm2

t_w, l'= wall thickness and clear span of installed wall panel, mm

α= reduction factor, 1 for shear column and 0.7 for flexural column

τ_0

Q_j=Sum of the shear strengths of connectors

underneath the beam

pQ_c=Direct shear strength of column

$=K_{min}.\tau_0.\ b_e.D$

K_{min}= 0.34/(.52+a/D)

b_e=effective width of columns, D=depth of columns,

τ_0= f(σ, F_{c1})

$\sigma=p_g.\ \sigma_y + \sigma_0$

p_g= ratio of a_g to b_e.D

σ_y=yield strength of longitudinal bars of a column

σ_0=N/b_e.D

Ultimate flexural strength

$wM_u = a_t.\ \sigma_{sy}.l_w + 0.5\sum(a_{wy}.\sigma_{wy}).l_w + 0.5Nl_w$

a_t, $\sum a_{wy}$ = cross sectional area of main bars of a boundary column and Vertical bars in the wall, respectively in mm^2

σ_{sy}, σ_{wy}= yield strength of longitudinal bars of a boundary column and Vertical bars in the wall, respectively (N/mm^2)

N= total axial force in the boundary columns

l_w= distance between the centre of the boundary columns of the wall, mm

Retrofitting with Shear Wall

Main features of Shear wall:

•This method is done for strength upgrading.

•Uplift strength of wall shall not be less than shear strength.

•Check structural balance i.e. eccentricity.

•Check capacity of solid shear wall as well as connections with boundary frame.

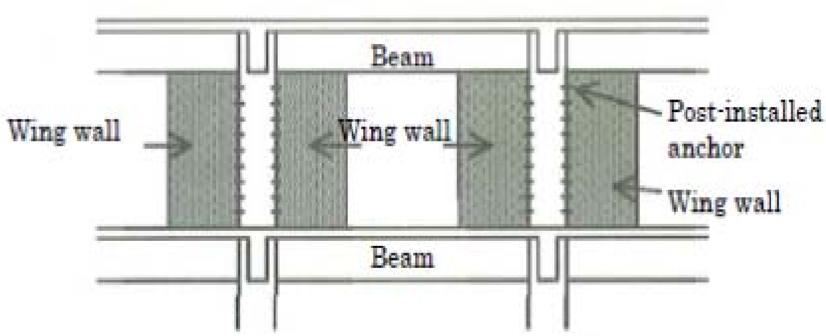
•Thickness of wall shall not be less than 15 cm but not more than the width of the beam.

•Reduce lighting and ventilation or subdivide inner spaces.

Meguro-ward Government Office

5: Constructing New RC Wing Wall to RC Column

Wing walls of reinforced concrete constructions are newly established in existing building columns



CAPACITY AFTER ADDING WING WALL:

Ultimate Flexural Strength

$$M_u=(0.9+\beta)a_t\sigma_y D+0.5ND\{1+2\beta-\frac{N}{\alpha_e b D F_{ct}}(\frac{a_t\sigma_y}{N}+1)^2\}$$

Ultimate Shear Strength

$$Q_{su}=\phi\{\frac{0.053\rho_{te}^{0.23}(F_c+18)}{\frac{M}{Q.d_e}+0.12}+0.85\sqrt{\rho_{we}\sigma_{wy}}+0.1\sigma_{0e}\}b_e j_e$$

$\alpha_e=(1+2\alpha\beta)/(1+2\beta)$

F_{c1} =compressive strength of concrete for wing wall , N/mm^2

N= axial force of column, N

b= width of column, D=depth of column,

a_t= gross sectional area of main bars of column in tensile side, mm^2

σ_y= yield strength of main bars of column, N/mm^2 and $\varphi=0.8$

F_c =compressive strength of concrete for existing structure, N/mm^2

$p_{te}= 100a_t/(b_e.d_e)$ $b_e=\alpha_e. b$ in mm

$p_{we}. \sigma_{wy}= p_w, \sigma_{wy}(b/b_e)+ p_{sh}, \sigma_{sy}(t/b_e)$

p_w, σ_{wy}= hoop ratio and yield strength of existing column, N/mm^2

p_{sh}, σ_{sy}=lateral reinforcement ratio of installed wing wall and its yield strength ,N/mm^2

$\sigma_{oe}= N/b_e. j_e$ and $j_e= 7d_e/8$ in mm

Retrofitting with Wing Wall

Main features of Wing wall:

•This method is usually done for strength upgrading.

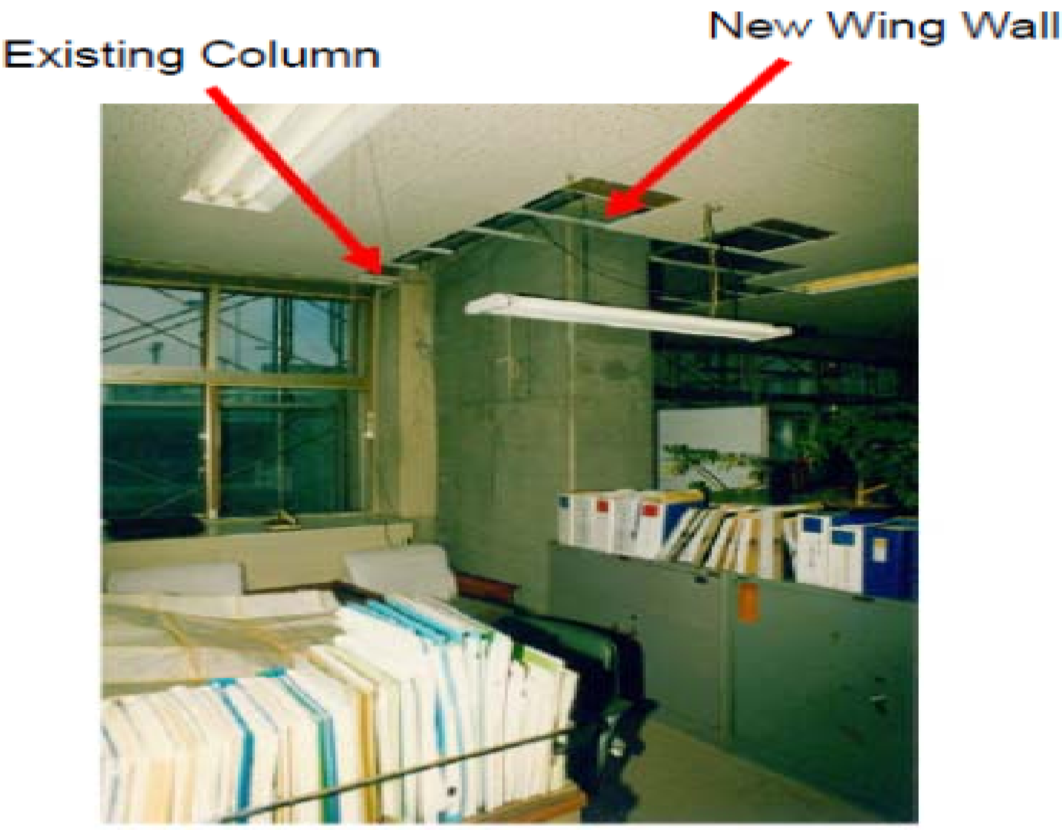
•Seismic performance may be upgraded by changing failure mechanism from column yielding to beam yielding

.

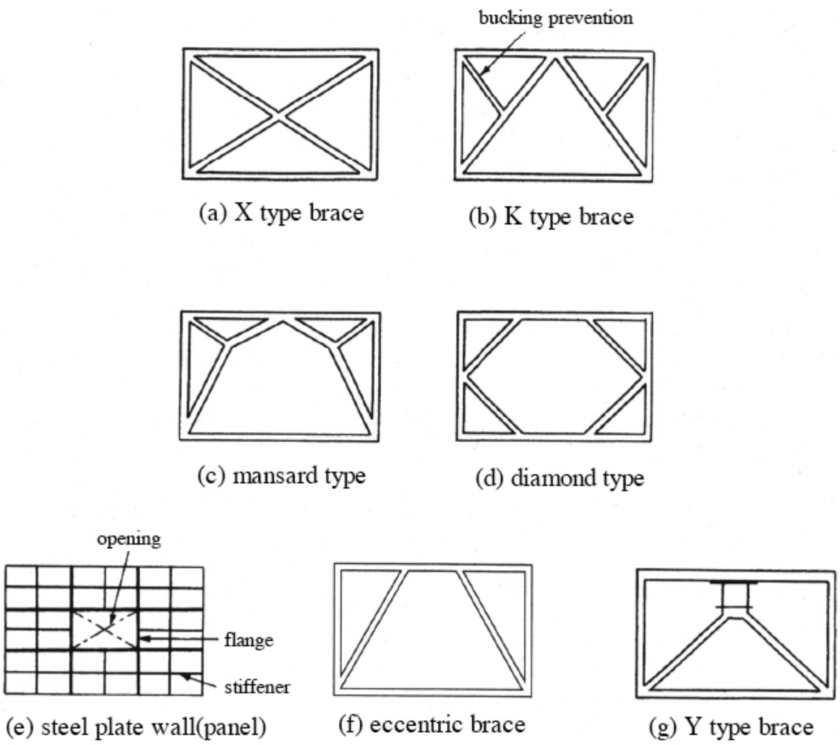
•Not suitable for column with short span beam, ensure clear span/depth ratio is more than 4.

•Check structural balance i.e. eccentricity.

•Thickness of wall shall not be less than 20 cm.



Retrofitting with Steel Frame



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Steel bracing:

•Steel member

•Connection

•Headed stud

•Post installed anchor

F = 1.5 to 2.0 subject to failure mode of steel bracing , connection and RC frame

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Retrofitting with Steel Frame

Main features of Steel frame:

•Steel framed braced/panel or non-frame brace/panel is inserted into existing RC frame.

•Resistance mechanism after retrofitting may be-(1) strength dominant type, (2) ductility dominant type (3) strength and ductility dominant type.

•Check structural balance i.e. eccentricity.

•Check local buckling of steel member.

•Check capacity of post installed anchor and studs.

•Lighting and ventilation is not so disturbed.

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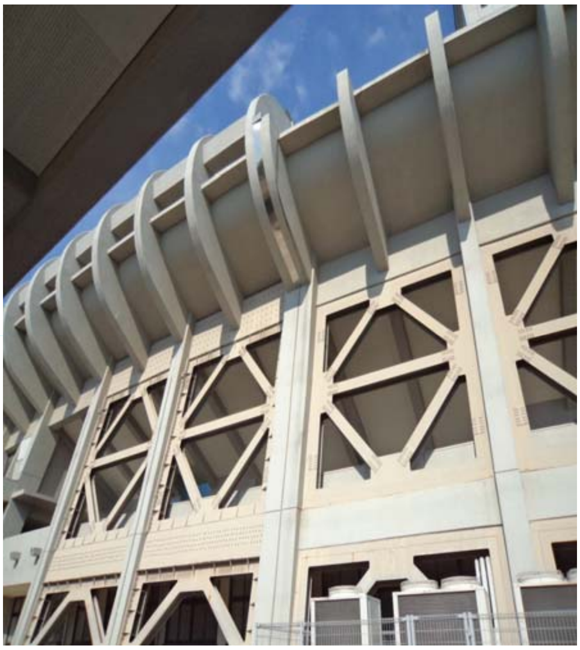

A stadium building of Japan

A school building of Japan

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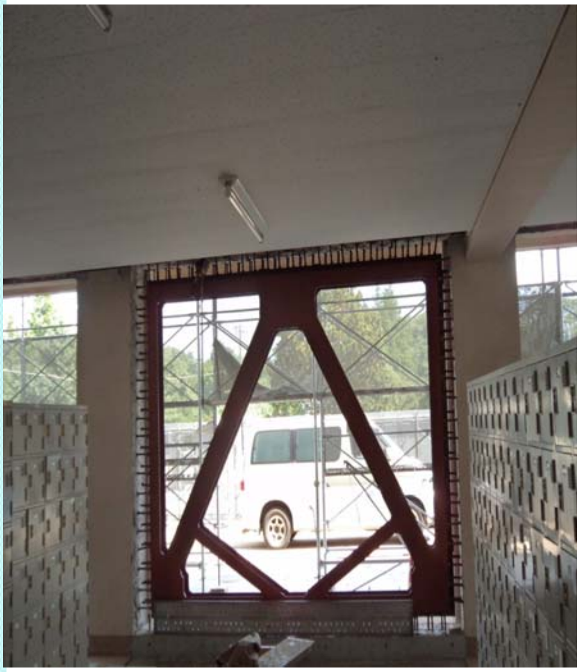

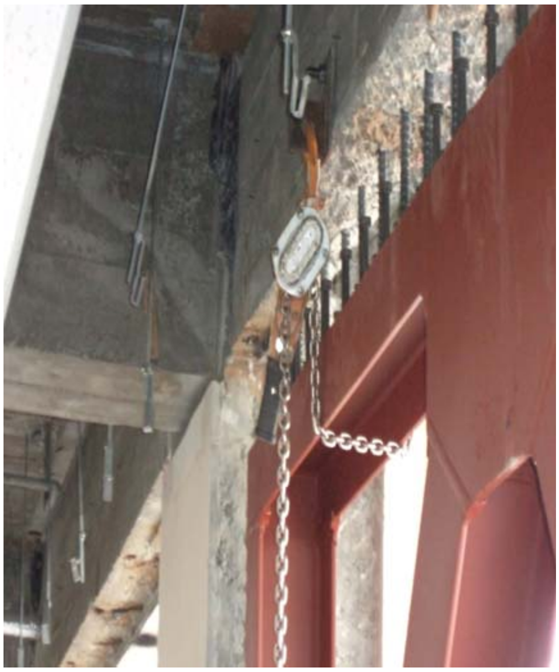

Retrofitting of a school building in Japan

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An Example of Seismic Evaluation and Retrofitting of Existing RC Buildings

2nd **level seismic screening** is applied, assuming column collapse.

General View of an Essential Hospital in Algiers, Algeria

Source: lecture from Akira INOUE, JICA Expert Team delivered on 09/June/2011

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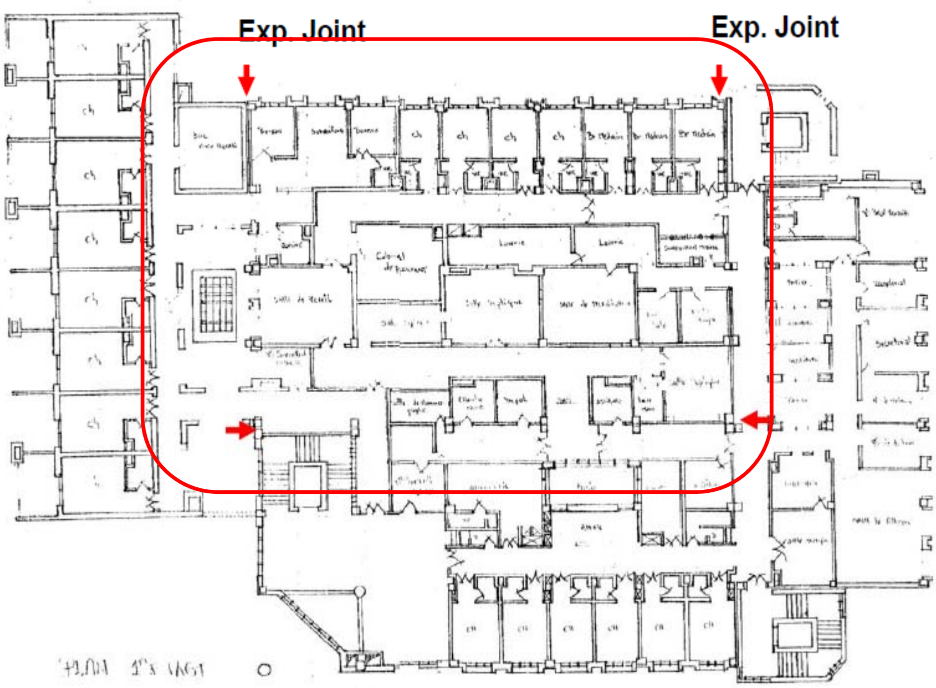
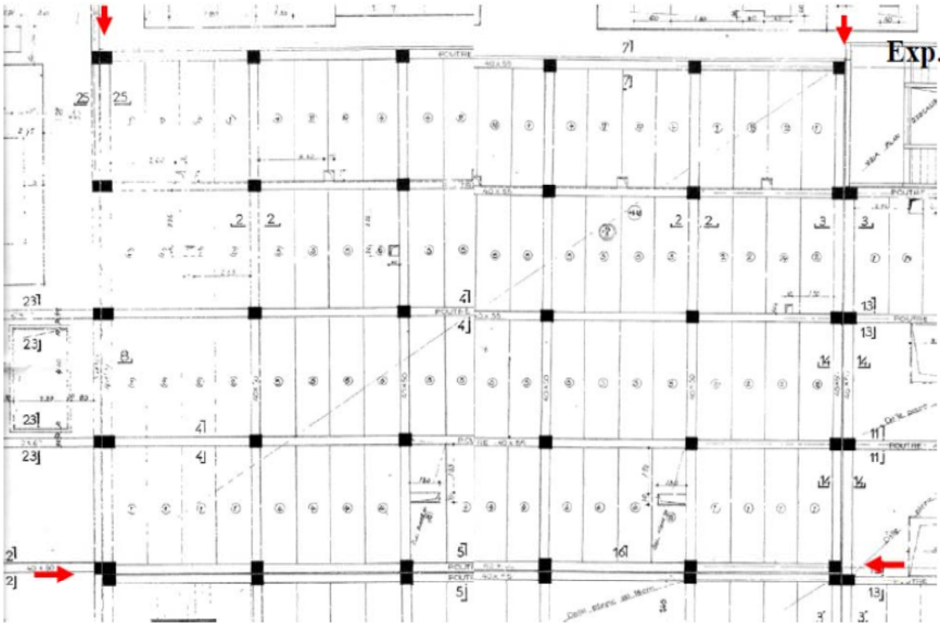
Architectural Plan (Ground floor)

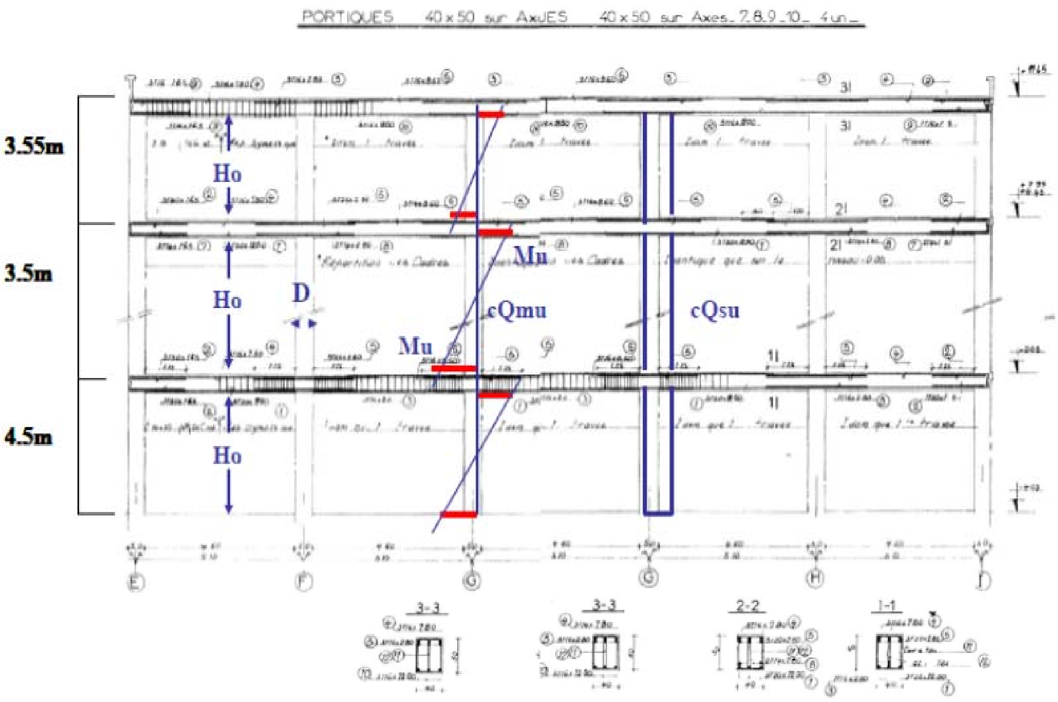


Structural Framing Plan (1st Floor)

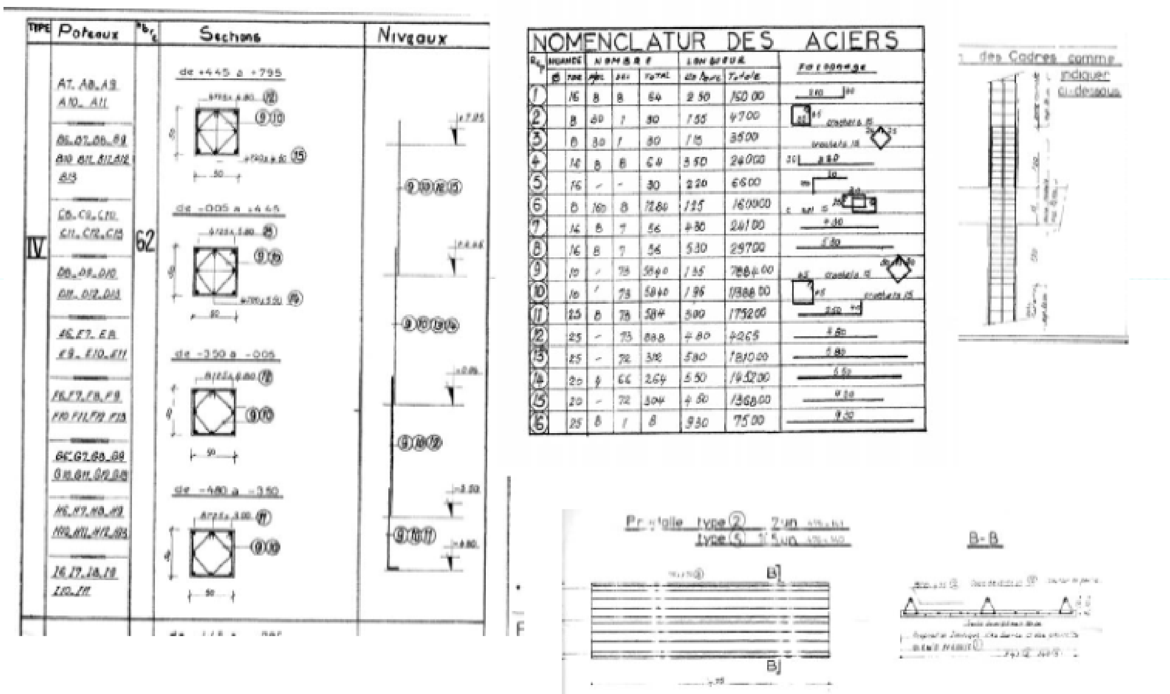
One Block of Moment Frame Building is Selected for Seismic Evaluation and Retrofitting



Structural Framing Elevation and Beam Section- design drawing



Column Section and Floor Slab System- design drawing



Building Dimensions, Weight of Building and Materials

- **Building Dimensions**
- X direction 6.0m span x 5=30m, Y direction 5.1m x 4=20.4m (grid line)
- Storey Height GF 4.5m, 1F 3.5m, 2F 3.55m total 11.55m
- Clear Length of Column Y direction 1F 4.0m, 2F 3.0m, 3F 3.05m

- **Unit Weight per Floor Area (Supposed Condition)**
- Roof 11 kN/m^2 (1.12tf/m^2)
- 1st Floor, 2nd Floor 14 kN/m^2 (1.43tf/m^2)

- **Weight of Building**
- Roof 7012kN
- 2nd Floor 8924kN 15936kN (Roof + 3rd floor)
- 1st Floor 8924kN 24860kN (Roof + 3rd + 2nd floor)

- **Material** (from Design Drawings)
- Re-bar Main Bars High Strength 412N/mm^2 (4200kg/cm^2) $\Phi \leqq 20$nn
- 392N/mm^2 (4000kg/cm^2) $\Phi > 20$nn
- Hoops, Stirrups Mild Steel 235N/mm^2 (2400kg/cm^2)
- Concrete (28 days strength) 27N/mm2 (275kg/cm^2)

Strength Index C, Flexural Strength of Columns

For $N_{max} \geq N > 0.4b \cdot D \cdot F_c$

$$M_u = \left\{0.8a_t \cdot \sigma_y \cdot D + 0.12b \cdot D^2 \cdot F_c\right\} \cdot \left(\frac{N_{max} - N}{N_{max} - 0.4b \cdot D \cdot F_c}\right)$$

For $0.4b \cdot D \cdot F_c \geq N > 0$

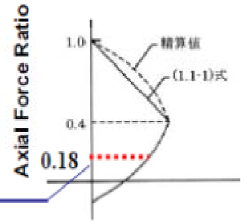
$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.5N \cdot D \cdot \left(1 - \frac{N}{b \cdot D \cdot F_c}\right) \quad (\text{N} \cdot \text{mm})$$

GF column=0.18

For $0 > N \geq N_{min}$

$$M_u = 0.8a_t \cdot \sigma_y \cdot D + 0.4N \cdot D$$

(A1.1-1)



Flexural Strength of Column

where:

N_{max} = Axial compressive strength $= b \cdot D \cdot F_c + a_g \cdot \sigma_y$ (N).

N_{min} = Axial tensile strength $= -a_g \cdot \sigma_y$ (N).

N = Axial force (N).

a_t = Total cross sectional area of tensile reinforcing bars (mm²).

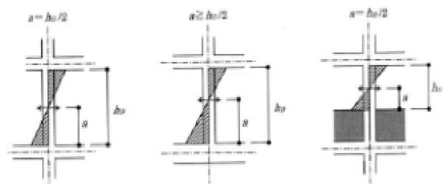
a_g = Total cross sectional area of reinforcing bars (mm²).

b = Column width (mm).

D = Column depth (mm).

σ_y = Yield strength of reinforcing bars (N/mm²).

F_c = Compressive strength of concrete (N/mm²).



Clear Height of Column

(b) The multi layered reinforcement shall be considered in using Eq. (A1.1-1).

(c) In calculating the ultimate flexural strength of columns, another calculation method su as based on rigid-plastic theory may be used instead.

Shear Strength of Columns

$$Q_{su} = \left\{\frac{0.053p_t^{0.23}(18 + F_c)}{M/(Q \cdot d) + 0.12} + 0.85\sqrt{p_w \cdot {}_s\sigma_{wy}} + 0.1\sigma_0\right\} \cdot b \cdot j \quad (\text{N}) \qquad (\text{A1.1-2})$$

where:

p_t = Tensile reinforcement ratio (%).

p_w = Shear reinforcement ratio, p_w =0.012 for $p_w \geq 0.012$.

${}_s\sigma_{wy}$ = Yield strength of shear reinforcing bars (N/mm²).

σ_0 = Axial stress in column (N/mm²).

d = Effective depth of column. D-50mm may be applied.

$\frac{M}{Q}$ = Shear span length. Default value is $\frac{h_0}{2}$.

h_0 = Clear height of the column.

j = Distance between centroids of tension and compression forces, default value is $0.8D$.

(b) If the value of $M/(Q \cdot d)$ is less than unity or greater than 3, the value of $M/(Q \cdot d)$ shall be unity or 3 respectively in using Eq. (A1.1-2). And if the value of σ_0 is greater than 8N/mm², the value of σ_0 shall be 8N/mm² in using Eq. (A1.1-2).

Flexural Strength of Column and Margin against Shear Failure

	Internal Columns				External Columns			
	Mu(kN・m)	cQmu(kN)	cQsn(kN)	cQsn/cQmu	Mu(kN・m)	cQmu(kN)	cQsn(kN)	cQmu/cQmu
2F	286	191	349	1.83	250	167	337	2.02
1F	375	250	384	1.53	303	202	356	1.76
GF	451	226	418	1.85	352	176	374	2.13

Mu: Ultimate Flexural Strength of Column (A1.1-1)

cQmu: Shear Force at the Ultimate Flexural Strength of Column $cQmu=Mu/(h_0/2)$

cQsn: Ultimate Shear Strength of Column (A1.1-2)

cQsn/cQmu: Strength Margin for Shear Failure of Flexural Column

Strength Index (C)

	ΣW(kN)	Internal Columns 12nos		External Columns 18nos		Total
		cQmux12	C=cQmux12/ΣW	cQmux18	C=cQmux18/ΣW	C
2F	7012	2288	0.326	3001	0.428	0.754
1F	15936	3002	0.188	3635	0.228	0.416
GF	24860	2707	0.109	3165	0.127	0.236

ΣW: Total Weight (Dead Load plus Live Load) Supported by the Storey Concerned

C: Strength Index C=Qu/ΣW (12)

Qu: Ultimate Lateral Load-carrying Capacity of the Vertical Members in the Storey Concerned, (in this case, Qu=cQmu(internal)x12+cQmu(external)x18)

Ductility Index (F) , Yield Deflection of Flexural Column

$R_{my} = (h_0 / H_0) \cdot {}_c R_{my} \geq R_{250}$ (A1.3-1)

where, $h_0 / H_0 \leq 1.0$ → $h_0/H_0=1.0$

→ ${}_c R_{my} = {}_c R_{150}$ for $h_0 / D \geq 3.0$ → $cRmy=cR_{150}=1/150$

${}_c R_{my} = {}_c R_{250}$ for $h_0 / D \leq 2.0$ (A1.3-2)

${}_c R_{my}$ is set by interporation for $2.0 < h_0 / D < 3.0$

where:

h_0 = Clear height of the column.

H_0 = Standard clear height of the column from the bottom of the upper floor beam to the top of the lower floor slab.

D = Column depth.

${}_c R_{150}$ = Standard drift angle of the column (measured in the clear height of column), 1/150.

${}_c R_{250}$ = Standard drift angle of the column (measured in the clear height of column), 1/250.

R_{250} = Standard inter-story drift angle, 1/250.

${}_c R_{my}$ = Yield drift angle of the column (measured in the clear height of column).

The value of ${}_c R_{my}$ shall not be greater than that of ${}_c R_{max}$ specified in the section 1.2(3) of Supplementary Provisions.

Plastic Drift Angle of Column

${}_c R_{mp} = 10({}_c Q_{su} / {}_c Q_{mu} - q) \cdot {}_c R_{my} \geq 0$ (A1.2-3) → cQsu/cQmu(Margin for Shear Failure)=1.83 to 2.13

→ $q = 1.0$ for $s \leq 100mm$

$q = 1.1$ for $s > 100mm$ (A1.2-4) → cRmp=0.055(1/18) to 0.075(1/13)

where:

${}_c Q_{su}$ = Ultimate shear strength of the column, calculated with Eq. (A1.1-2) in principle.

${}_c Q_{mu}$ = Shear force at the ultimate flexural strength of the column. The largest moment capacity shall be used under the working axial force, in case axial force of column is greater than the balanced axial force.

s = Spacing of hoops.

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Storey Drift Angle at Ultimate Flexural Strength of Column

$$R_{mu} = (h_0 / H_0) \cdot {}_cR_{mu} \geq R_{250} \quad \text{(A1.2-1)}$$

where, $h_0 / H_0 \leq 1.0$ → **h_0/H_0=1.0, Rmu=cRmu**

$${}_cR_{mu} = {}_cR_{my} + {}_cR_{mp} \leq {}_cR_{30} \quad \text{(A1.2-2)}$$

→ **cRmu=cRmy(1/150)+cRmp(A1.2-3)**

where:

h_0 = Clear height of column.

H_0 = Standard clear height of column from bottom of the upper floor beam to top of the lower floor slab.

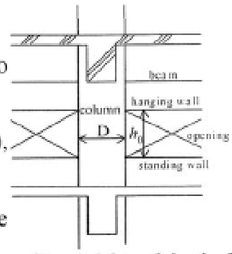
${}_cR_{my}$ = Yield drift angle of column (measured in clear height of column), specified in the section 1.3 of Supplementary Provisions.

${}_cR_{mu}$ = Drift angle at the ultimate flexural strength of column (measured in the clear height of column).

${}_cR_{mp}$ = Plastic drift angle of the column (measured in the clear height of column), specified in the section 1.2(2) of Supplementary Provisions.

${}_cR_{30}$ = Standard drift angle of the column (measured in the clear height of column), 1/30.

R_{250} = Standard inter-story drift angle, 1/250.



Clear height and depth of column

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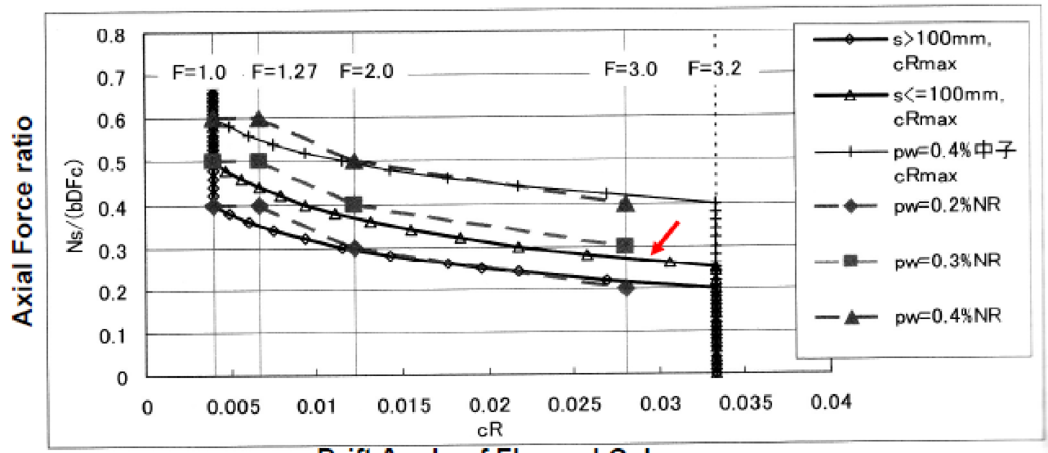
Upper Limit of Drift Angle of Column

- **(A1.2-5)**
- **1. Axial Force Ratio (A1.2-6)**
- **N/bDFc=0.18＜0.25 ----- cR30**
- **2. Shear Force (A1.2-7)**
- **cτu/Fc=0.042＜ 0.2 -----cR30**
- **3. Tensile Reinforcement Ratio (A1.2-8)**
- **Pt=0.52%＜1.0% ----cR30**
- **4. Spacing of Hoops (A1.2-8)**
- **S/db=@100/Φ20(Main Bar)=5＜8 ----cR30**
- **5. Clear Height of Column (A1.2-9)**
- **H0/D=3.0m(2F)/0.5m=6≧2 ----cR30**
- **Where, cR30: Standard Drift Angle of Column (measured in the**
- **clear height of column), 1/30**
- **→Ductility Factor μ=5.0 (upper limit is used)**

Upper Limit of Drift Angle of Flexural Column and Axial Force Ratio



Drift Angle of Flexural Column

Ductility Index (F) of Flexural Column

(i) In case $R_{mu} < R_y$ (Rmu)

$$F = 1.0 + 0.27\frac{R_{mu} - R_{250}}{R_y - R_{250}} \quad (15)$$

(ii) In case $R_{mu} \geq R_y$ (Rmu)

$$F = \frac{\sqrt{2R_{mu}/R_y - 1}}{0.75\cdot(1 + 0.05R_{mu}/R_y)} \leq 3.2 \quad (16)$$

→ Rmu/Ry=5.0, F=3.2

where:

R_y = Yield deformation in terms of inter-story drift angle, which in principle shall be taken as R_y=1/150.

R_{250} = Standard inter-story drift angle (corresponding to the ductility index of the shear wall), R_{250} = 1/250.

R_{mu} = Inter-story drift angle at the ultimate deformation capacity flexural failure of the column member, calculated by Eq. (A1.2 in the Supplementary Provisions 1.2(1).

Ductility-dominant Basic Seismic Index of Structure Eo

$$E_0 = \frac{n+1}{n+i}\sqrt{E_1^2 + E_2^2 + E_3^2}$$ **Eo=(3+1)/(3+1)・C1・F1 (GF)** (4)

where:

E_1 = $C_1 \cdot F_1$.
E_2 = $C_2 \cdot F_2$.
E_3 = $C_3 \cdot F_3$.
C_1 = The strength index C of the first group (with small F index).
C_2 = The strength index C of the second group (with medium F index).
C_3 = The strength index C of the third group (with large F index).
F_1 = The ductility index F of the first group.
F_2 = The ductility index F of the second group.
F_3 = The ductility index F of the third group.

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Strength-dominant Basic Seismic Index of Structure Eo

$$E_0 = \frac{n+1}{n+i}\left(C_1 + \sum_j \alpha_j C_j\right) \cdot F_1 \qquad (5)$$

where:

α_j = Effective strength factor in the j-th group at the ultimate deformation R_1 corresponding to the first group (ductility index of F_1), given in Table 3.

Table 3 Effective strength factor

Cumulative point of the first group F_1 =0.8 (Drift angle R_1=R_{500}=1/500)		
	F_1	F_1=0.8
	R_1	R_1=R_{500}
Second and higher groups	Shear (R_{su}=R_{250})	α_S
	Shear (R_{250}< R_{su})	α_S
	Flexural (R_{my}=R_{250})	0.65
	Flexural (R_{250}<R_{my}<R_{150})	α_m
	Flexural (R_{my}=R_{150})	0.51
	Flexural and shear walls	0.65

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Irregularity Index S_D Table 6

Expansion Joint 4cm/1150cm(height)=1/287.5＜1/200, 1/100 q2d=0.95
Storey Height Uniformity i=3.5m(1F)/4.5m(GF)=0.78＜0.8 q2i=0.975 S_D=0.926

Table 6 Classification of items and G, R-values

			Gi (Grade)			R (adjustment fator)	
			1.0	0.9	0.8	R1i	R2i
Horizontal balance	a	Regularity	Regular a1	Nearly regular a2	Irregular a3	1.0	0.5
	b	Aspect ratio of plan	b≦5	5＜b≦8	8＜b	0.5	0.25
	c	Narrow part	0.8≦c	0.5≦c＜0.8	c＜0.5	0.5	0.25
	d	Expansion joint *1	1/100≦d	1/200≦d＜1/100	D＜1/200	0.5	0.25
	e	Well-style area	e≦0.1	5＜e≦8	0.3＜e	0.5	0.25
	f	Eccentric well-style area*2	f_1≦0.4 & f_2≦0.1	f_1≦0.4 & 0.1＜f_2≦0.3	0.4＜f_1 or 0.3＜f_2	0.25	0
	g						
Elevation balance	h	Underground floor	1.0≦h	0.5≦h＜1.0	h＜0.5	0.5	0.5
	i	Story height uniformity	0.8≦I	0.7≦I＜0.8	I＜0.7	0.5	0.25
	j	Soft story	No soft story	Soft story	Eccentric soft story	1.0	1.0
	k						
Eccentricity	l	Eccentricity*3	l≦0.1	0.1＜l≦0.15	0.15＜l		1.0
	m						1.0
Stiffness	n	(Stiffness/mass)Ratio of above and below stories	n≦1.3	1.3＜n≦1.7	1.7＜n		1.0
	o						1.0

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Time Index T=0.95 (assumed) Table 8

Table 8 Evaluation of time index by the second level inspection (-story) for the second level screening

Portion	Range (Item / Degree)	Structural cracking and deflection a	b	c	Deterioration and aging a	b	c
	Degree	1. Cracking caused by uneven settlement. 2. Shear or inclined cracking in beams, walls, and/or columns, observed evidently.	1. Deflection of a slab and/or beam, affecting on the function of non-structural element. 2. Same as left but not visible from some distance. 3. Same as above but can be observed from some distance.	1. Minute structural cracking not corresponding to the items a or b. 2. Deflection of a slab and/or beam, not corresponding to the item a or b.	1. Cracking by concrete expansion due to the rust of reinforcing bar. 2. Rust of reinforcing bar. 3. Cracking caused by a fire disaster. 4. Deterioration of concrete caused by chemicals.	1. Seep of the rust of reinforcing bar due to rain water or water leak. 2. Neutralization to the depth of reinforcing bar or equivalent aging. 3. Spalling off of finishing materials.	1. Remarkable blemish of concrete due to rain water, water leak, and chemicals. 2. Deterioration or slight spalling off of a finishing material.
I Slab including sub-beam	1) 1/3 or more of total floor	0.017	0.005	0.001	0.017	0.005	0.001
	2) 1/3~1/9	0.006	0.002	0	0.006	0.002	0
	3) 1/9 or less	0.002	0.001	0	0.002	0.001	0
	4) 0 #	0	0	0	0	0	0
II Beam	1) 1/3 or more of total number of members for each direction	0.05	0.015	0.004	0.05	0.015	0.004
	2) 1/3~1/9	0.017	0.005	0.001	0.017	0.005	0.001
	3) 1/9 or less	0.006	0.002	0	0.006	0.002	0
	4) 0 #	0	0	0	0	0	0
III Wall & Column	1) 1/3 or more of total number of members	0.15	0.045	0.011	0.15	0.045	0.011
	2) 1/3~1/9	0.05	0.015	0.004	0.05	0.015	0.004
	3) 1/9 or less	0.017	0.005	0.001	0.017	0.005	0.001
	4) 0 #	0	0	0	0	0	0
Mark-down	Subtotal						
Total	Ground total	P1			P2		

Remark(#). The item 4) may be adopted in case where there are no areas or members with aging defect, and the maintenance condition of the building could be recognized as very good.

2nd Level Seismic Screening
Seismic Index of Structure, Is, Y Direction

	C	F	n+1/n+i	Eo	S_D	T	Is
2F	0.76	3.2	0.67	1.63	1.11*	0.95	1.72*
1F	0.42	3.2	0.80	1.07	1.11*	0.95	1.13*
GF	0.24	3.2	1.00	0.76	1.00*	0.95	0.72*

C: Strength Index (12), (A1.1-1), (A1.1-2)

F: Ductility Index (16)

n: Number of storey

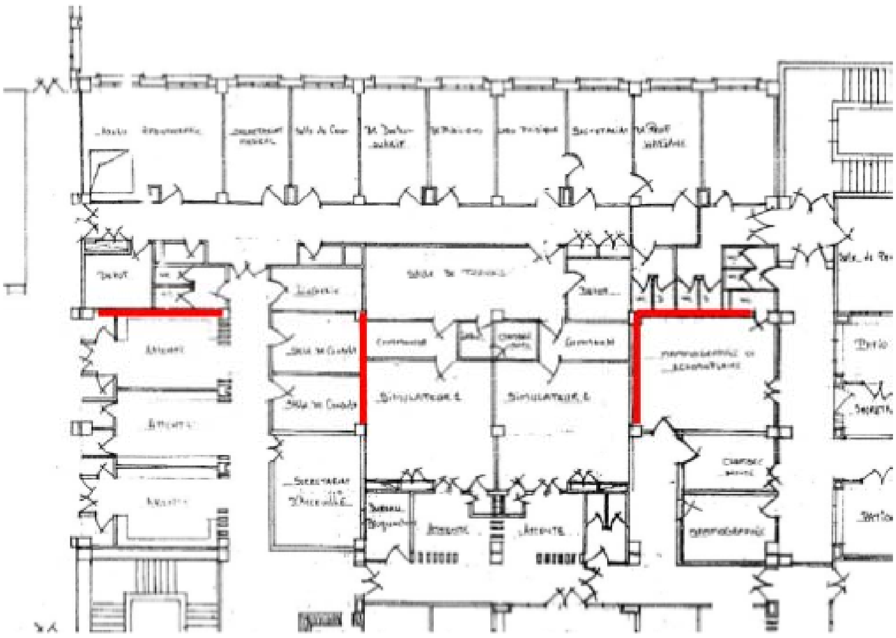
Eo: Basic Seismic Intensity of Structure Eo=(n+1/n+i)CF (4)

S_D: Irregularity Index (Expansion Joint x0.95, Storey Height Uniformity x0.975, Underground Floor x1.20*, Stiffness/mass Ratio x1.0(3F,2F), 0.9(1F)*) Table 6

T: Time Index (0.95 is used) Table 8

Is: Seismic Index of Structure Is=EoS$_D$T (1)

Case 1: Retrofit by RC Walls, 1st Storey



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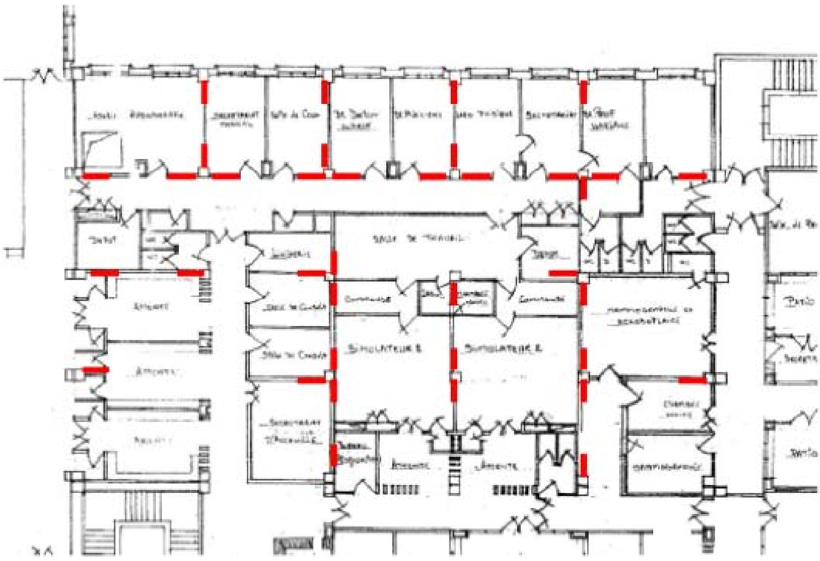
Case 2: Retrofit by Wing-walls, Ground Storey



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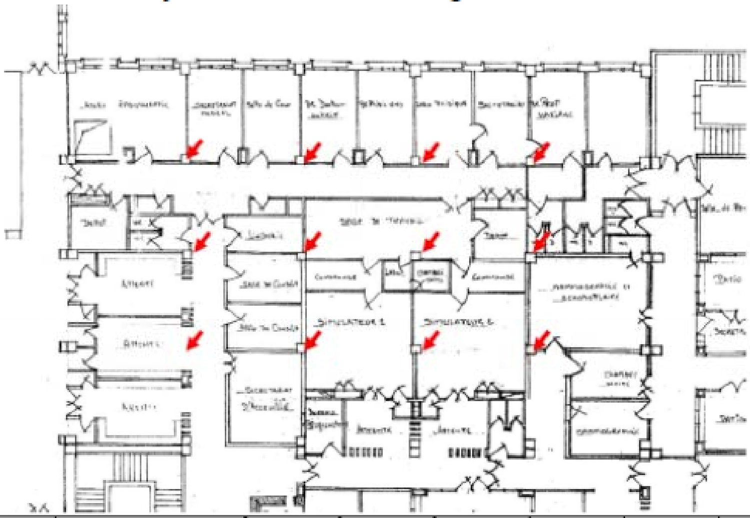
Case 3: Retrofit by Column Jacketing, Ground floor

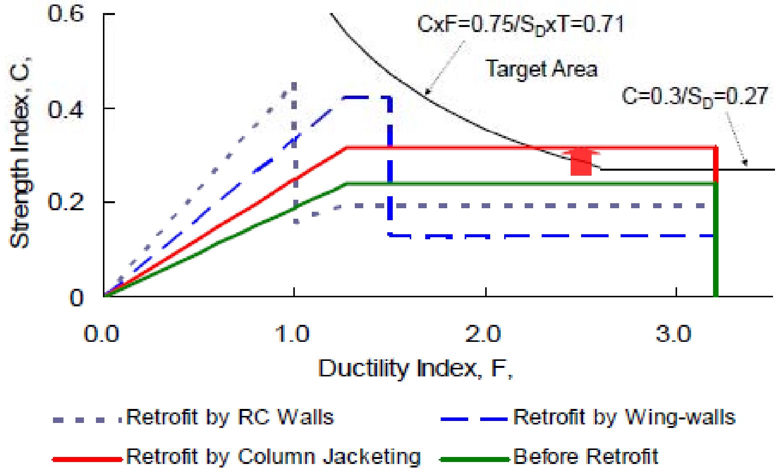


Direction	C		Column	Total C	F	Eo(4)	n+1/n+i	Is	$C_T S_D$
X	0.00708	(perimeter)	18	0.3158	3.2	1.01	1.0	1.07	0.35
	0.0157	(center)	12						
Y	0.00708	(perimeter)	18	0.3134	3.2	1.00	1.0	1.06	0.35
	0.0155	(center)	12						

S_D=1.11, T=0.95 **Strength increase 0.316/0.24=1.32**

Strength Index (*C*) and Ductility Index (*F*)

in X direction of 1st Storey, column jacketing is proposed



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THANK YOU

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Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

Retrofitting Works

Md. Sohel Rahman
Executive Engineer
PWD Design Division-4
and
Team Leader, Component 3
CNCRP Project

Methods of Retrofitting

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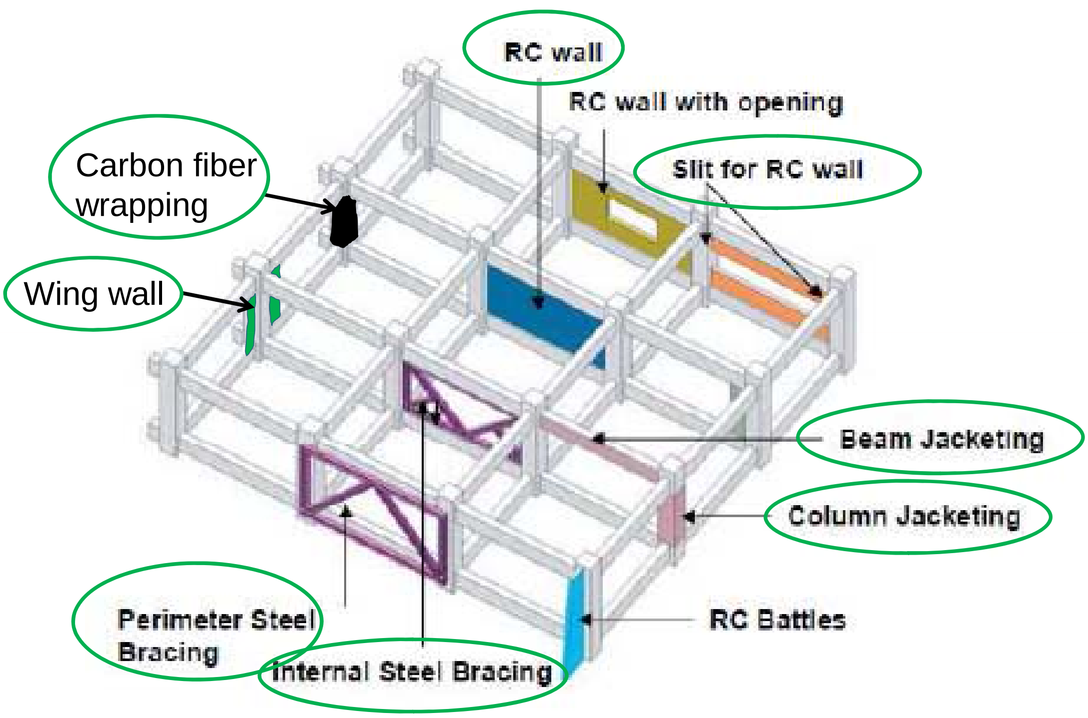


Retrofitting with Column Jacketing

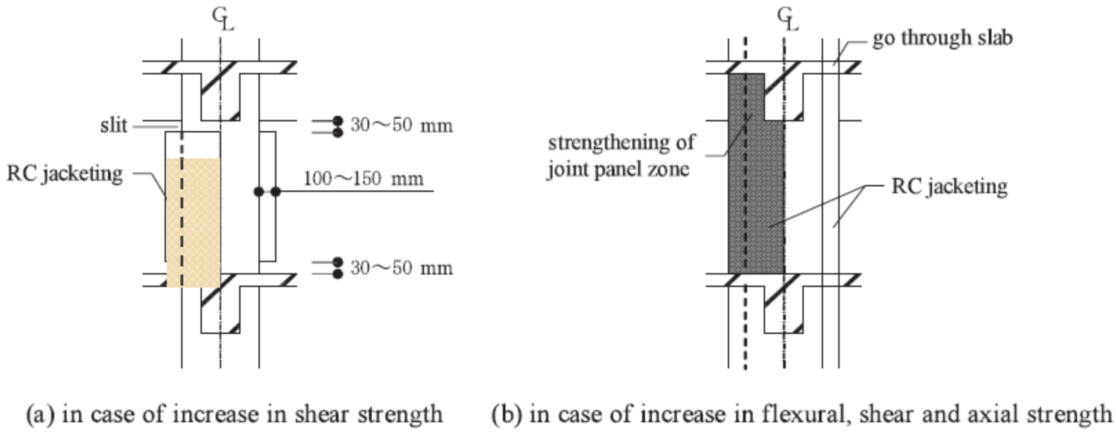
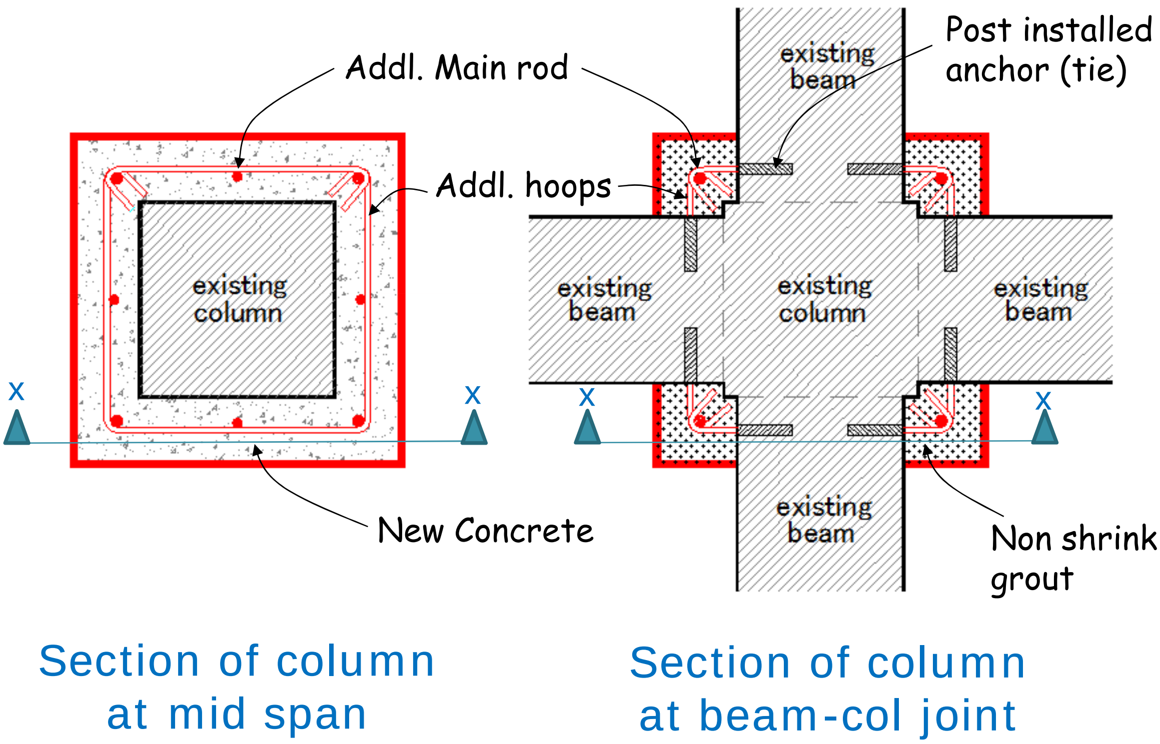


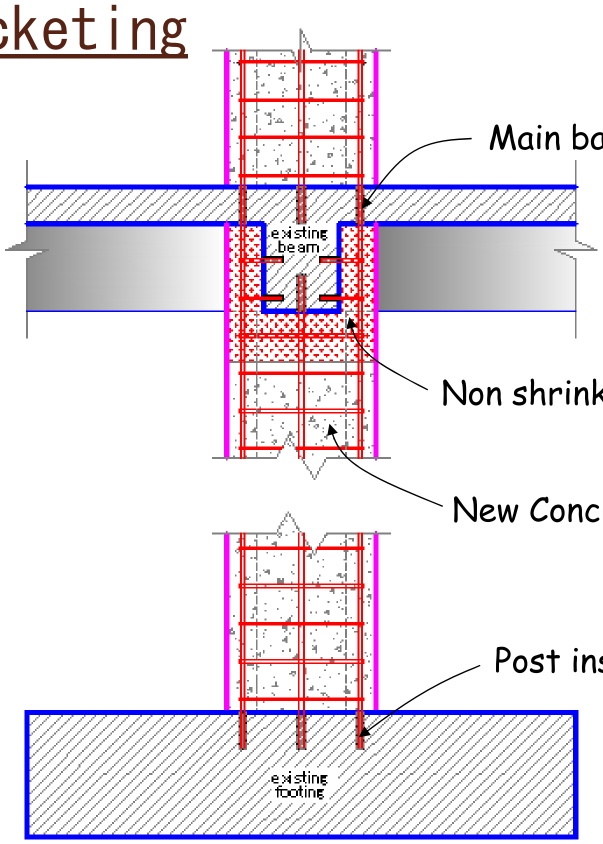
Figure TN.16 Column strengthening with RC jacketing

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Retrofitting with Column Jacketing



Retrofitting with Column Jacketing



Section of column jacketing

Retrofitting with Column Jacketing

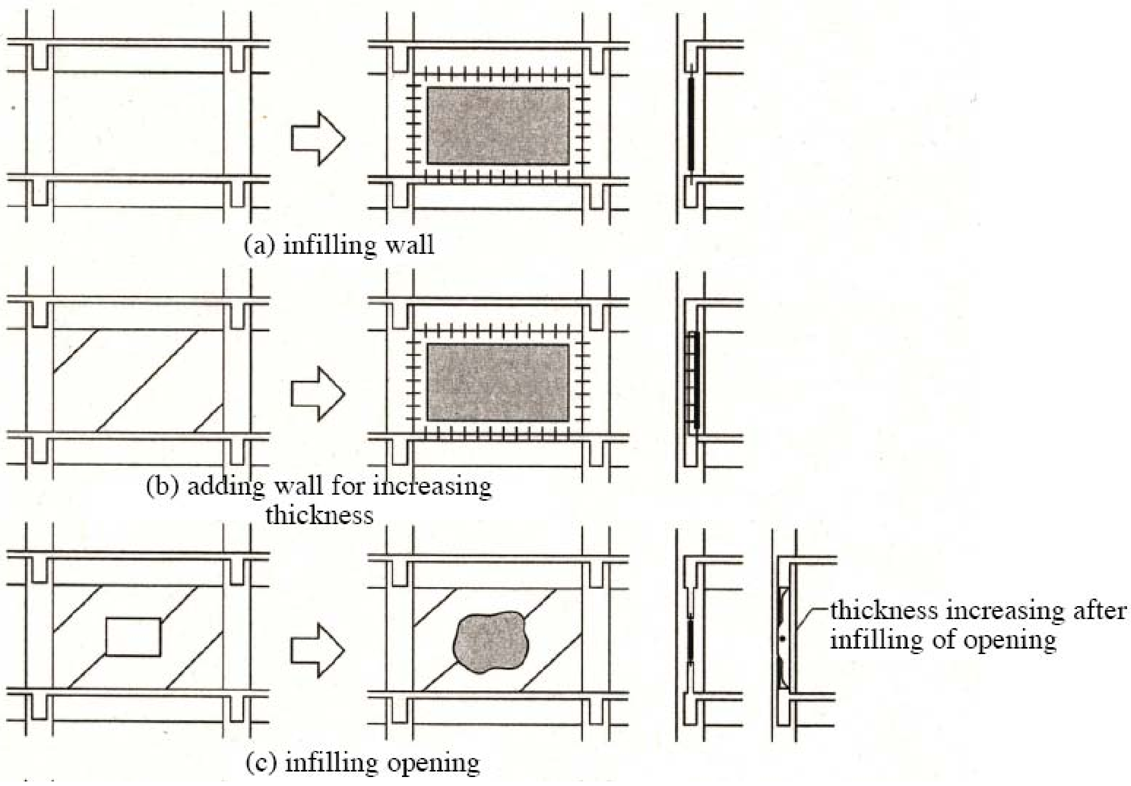
RC Column Jacketing through beam-col joint

RC Column Jacketing

Test Work of CNCRP in 2012

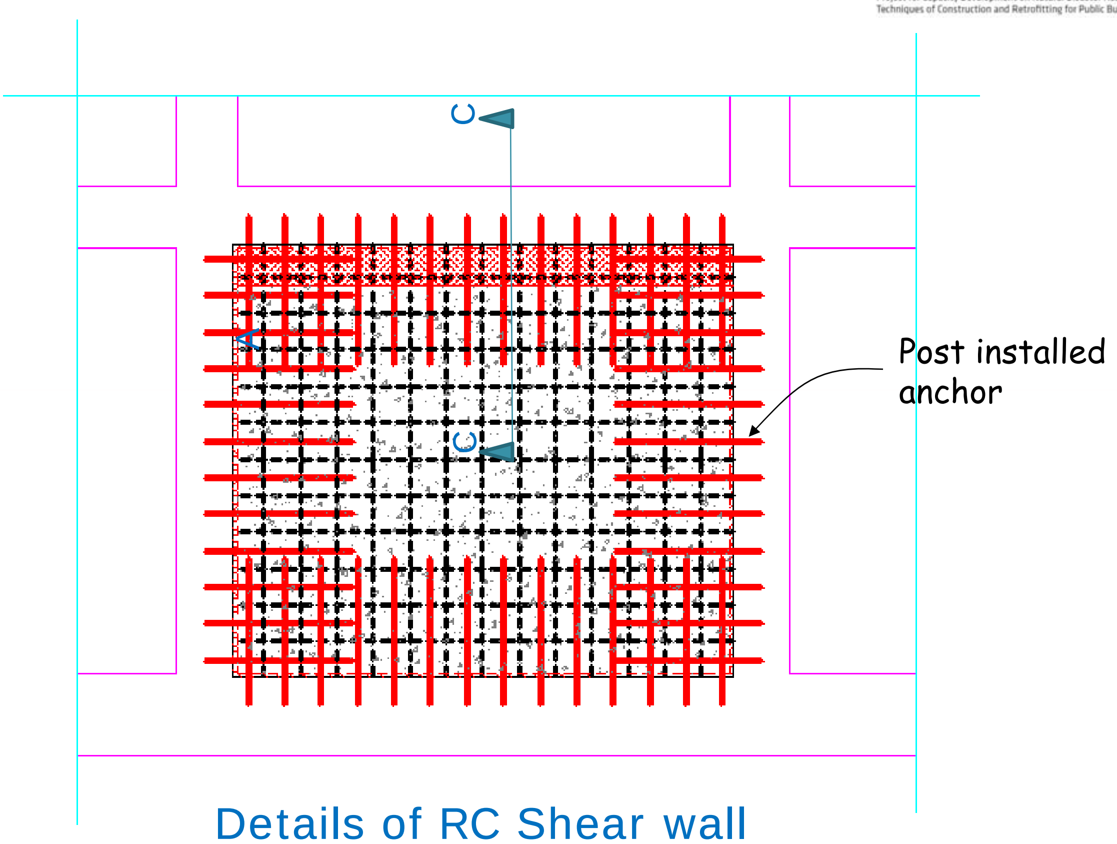
Test Work of CNCRP in 2013

Retrofitting with Shear Wall



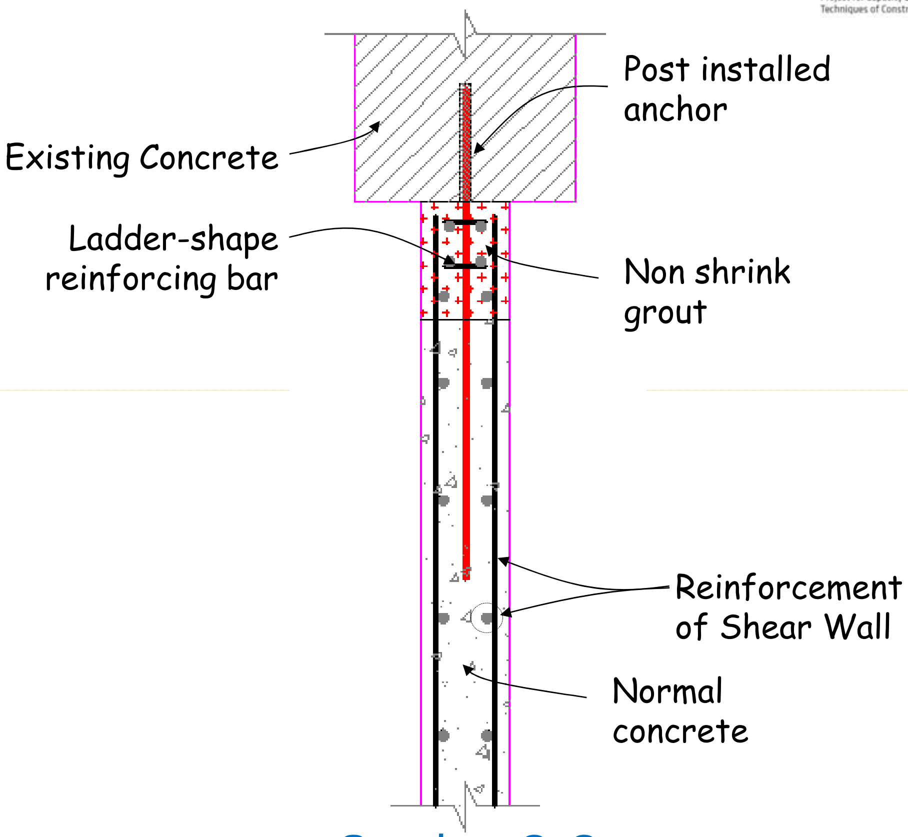
SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Retrofitting with Shear Wall



Details of RC Shear wall

Retrofitting with Shear Wall



Section C-C

Retrofitting with Shear Wall

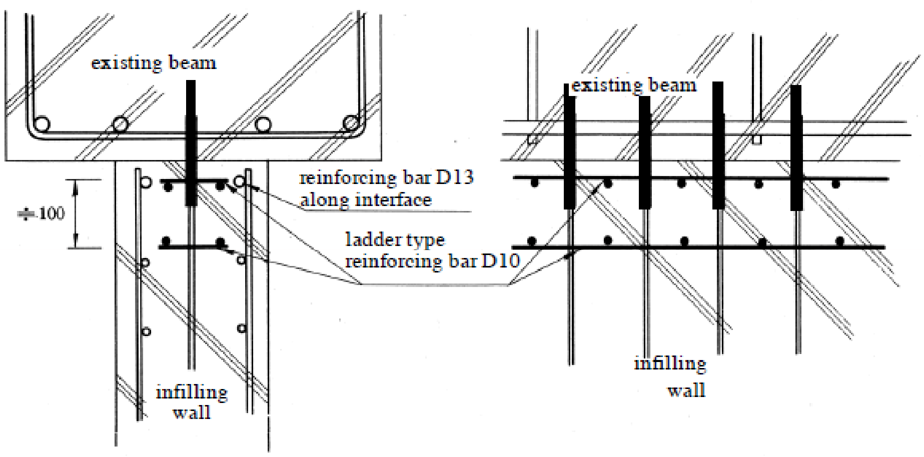


Figure TN.12 Strengthening against splitting with ladder type reinforcing bars
(quoted from the figure on page 98 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Retrofitting with Shear Wall

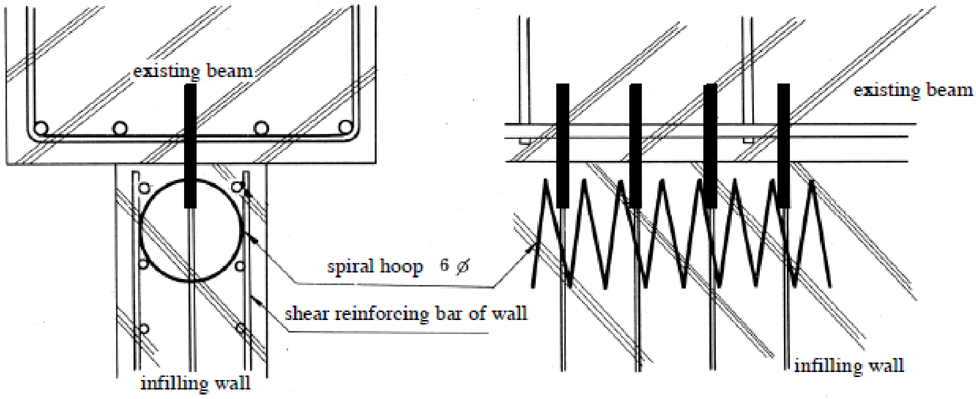
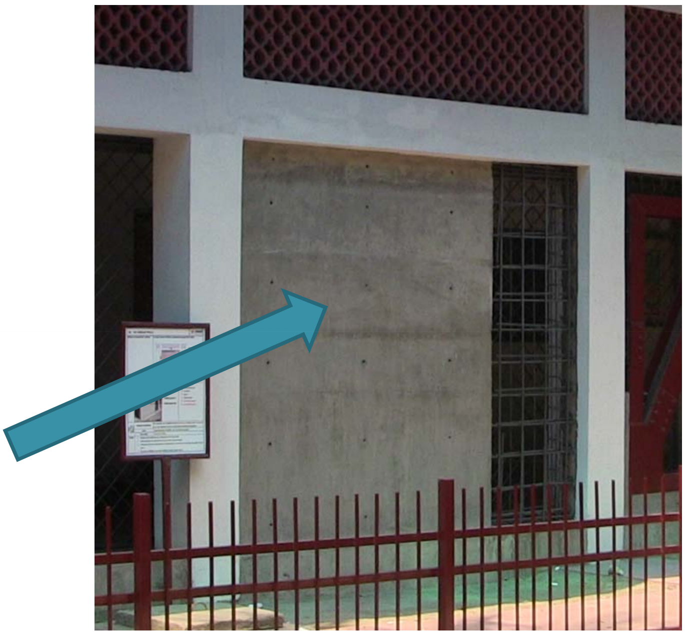


Figure TN.11 Strengthening against splitting with spiral reinforcing bars
(quoted from the figure on page 98 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

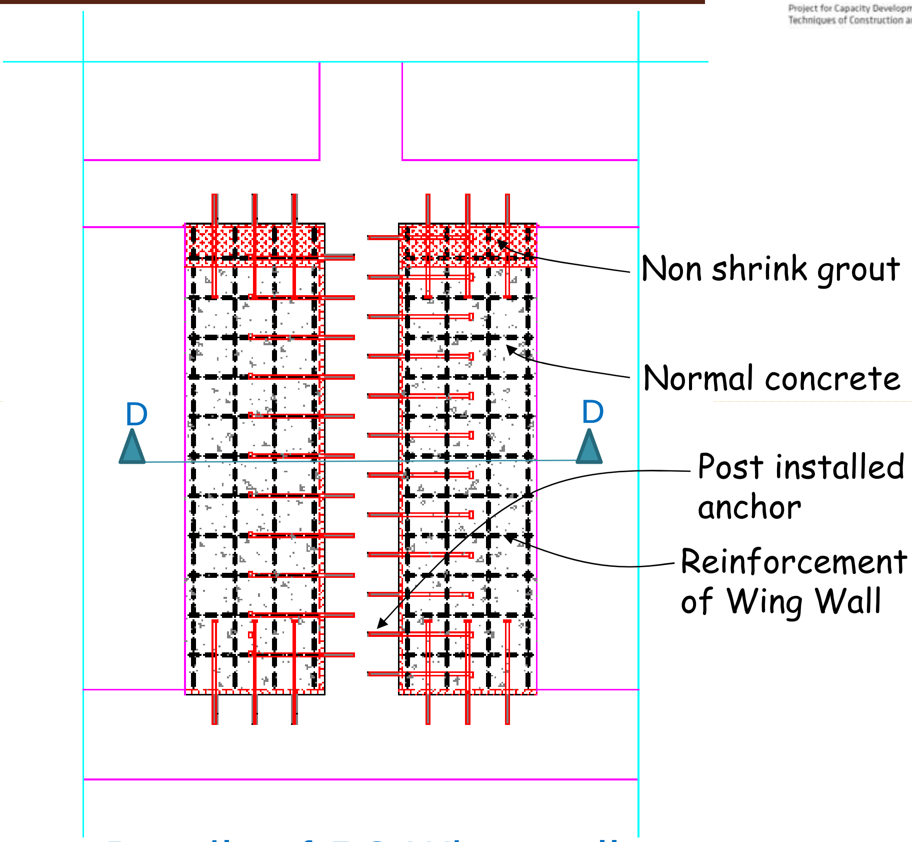
Retrofitting with Shear Wall

Providing RC Shear Wall in a open frame



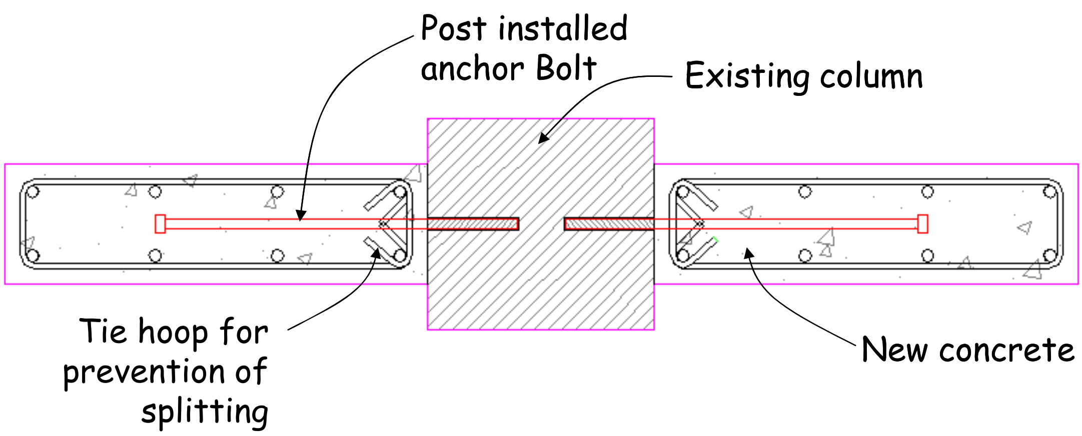
Test Work of CNCRP in 2012

Retrofitting with Wing Wall



Details of RC Wing wall

Retrofitting with Wing Wall



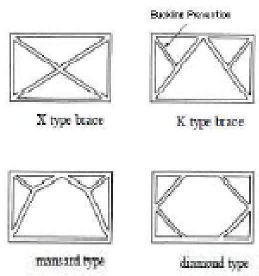
Section D-D

Retrofitting with Wing Wall

Test Work of CNCRP in 2012

Retrofitting with Steel Frame

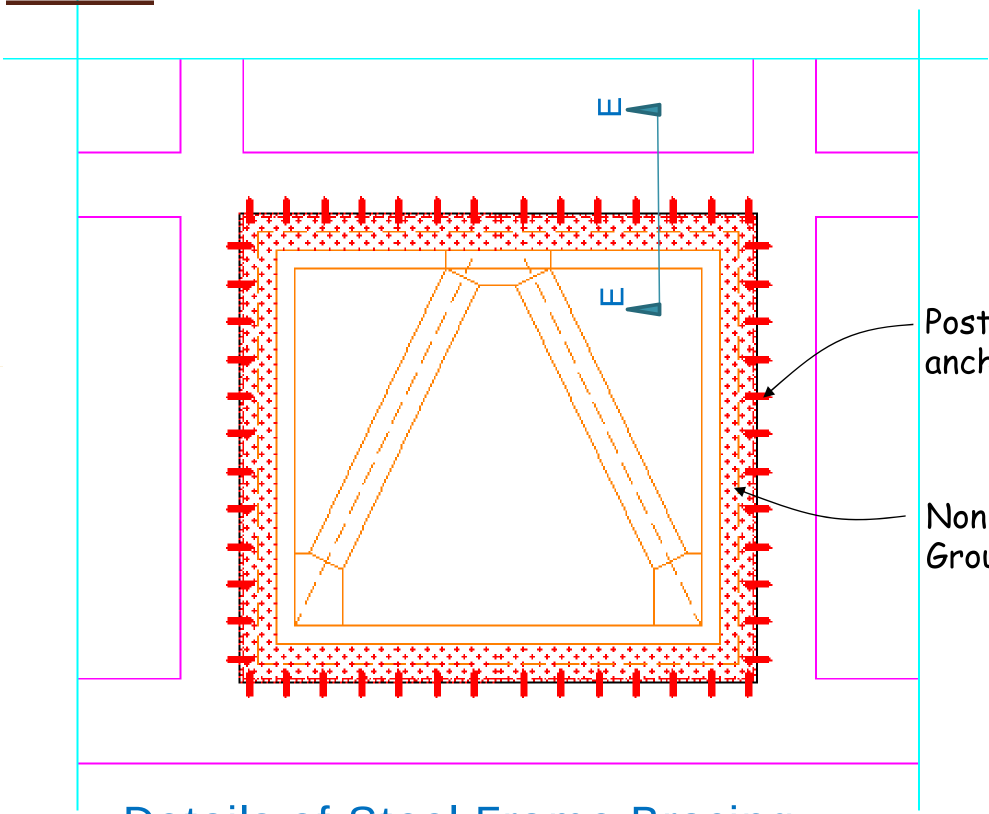
1. Steel Framed Bracing



Ookashiwa Elementary School

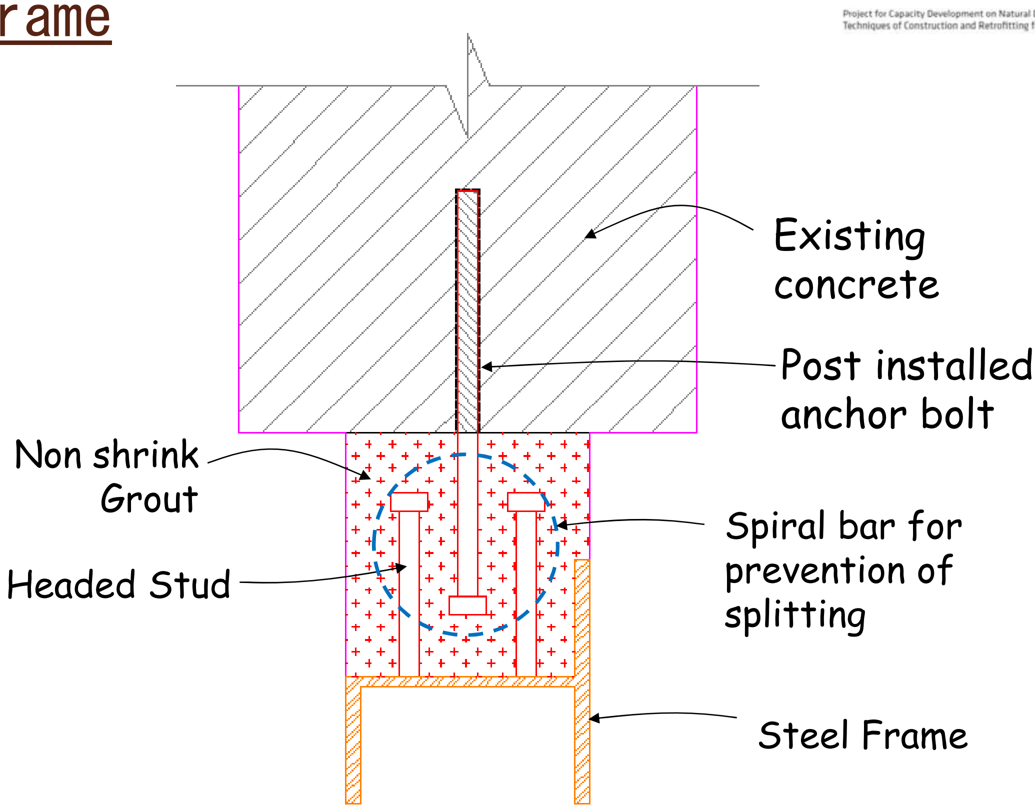
SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Retrofitting with Steel Frame



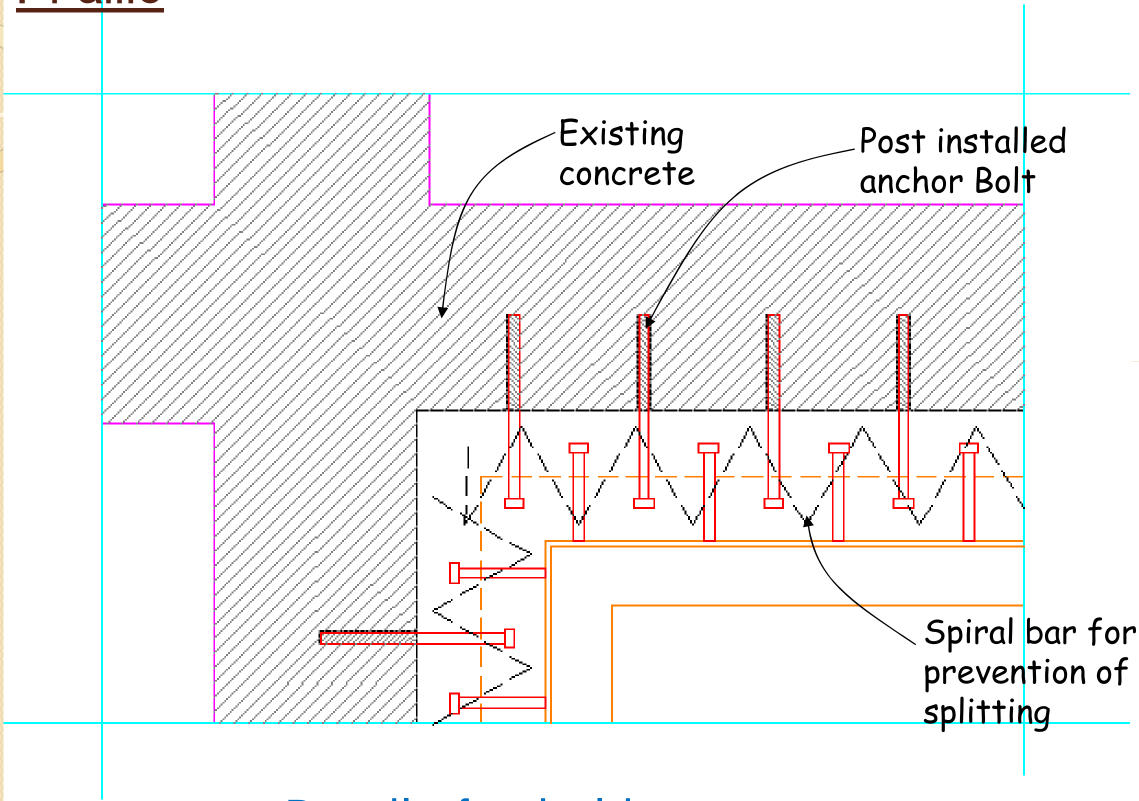
Details of Steel Frame Bracing

Retrofitting with Steel Frame



Section E-E

Retrofitting with Steel Frame



Detail of spiral bar

Retrofitting with Steel Frame

Test Work of CNCRP in 2012

Internal Steel Frame Bracing

External Steel Frame Bracing

Connection Details, Test Work 2012

Test Work of CNCRP in 2013

Steel framed bracing

a) Junior High School Building (School Colored)

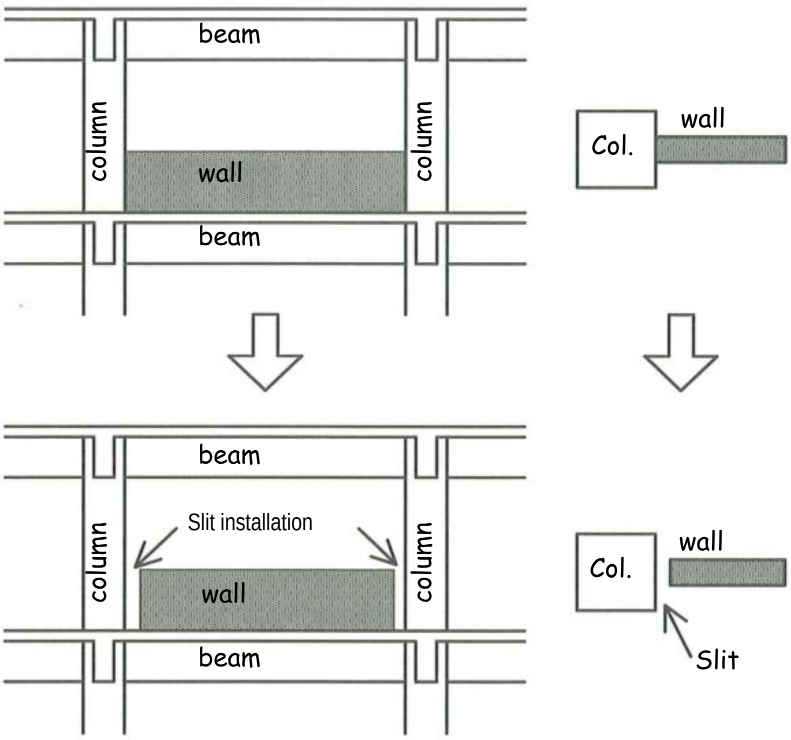
b) Junior High School Building (Designed by Students)

c) University Building (with Exterior Panels)

d) Elementary School Building (with Balconies)

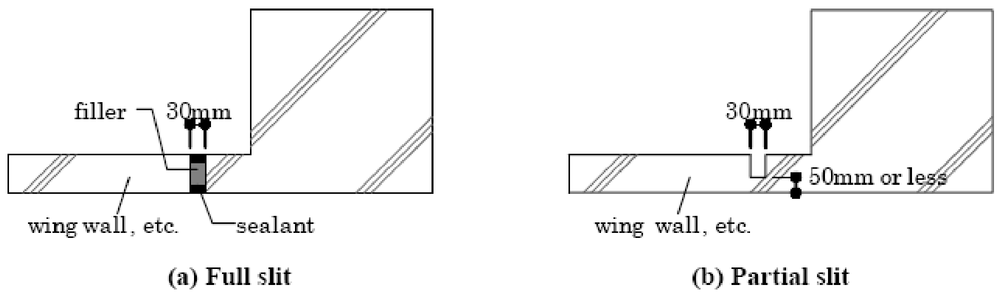
SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Retrofitting with Structural Slit



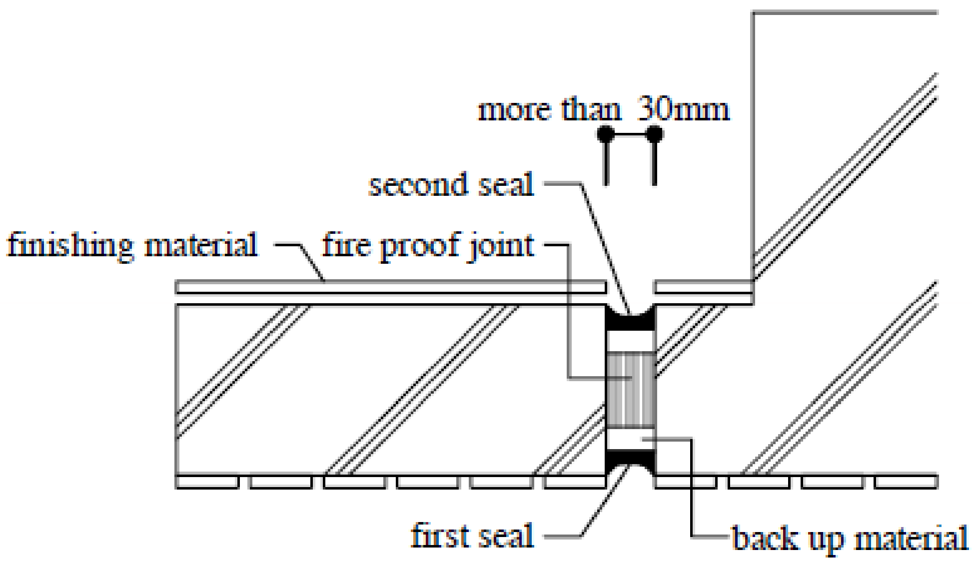
SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Retrofitting with Structural Slit



SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Retrofitting with Structural Slit



Detail of Seismic Slit

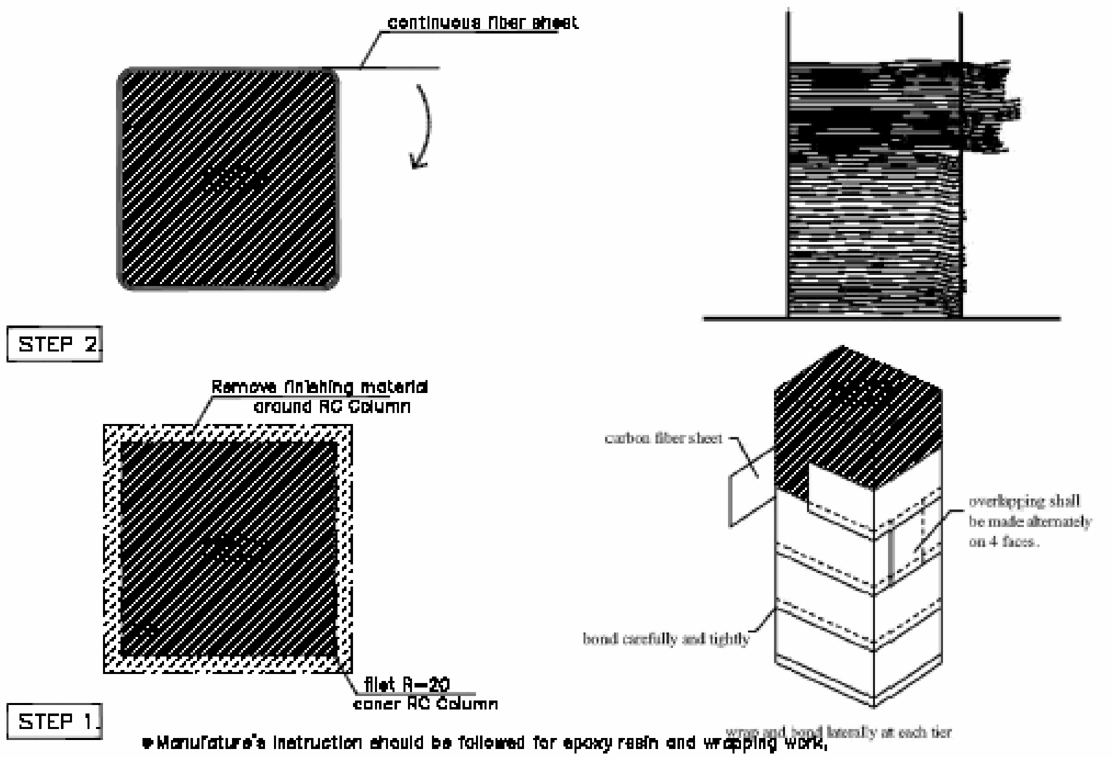
SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Retrofitting with Structural Slit

Seismic Slit is provided at a brick wall

Test Work of CNCRP in 2012

Carbon fiber sheet wrapping



Carbon fiber sheet wrapping

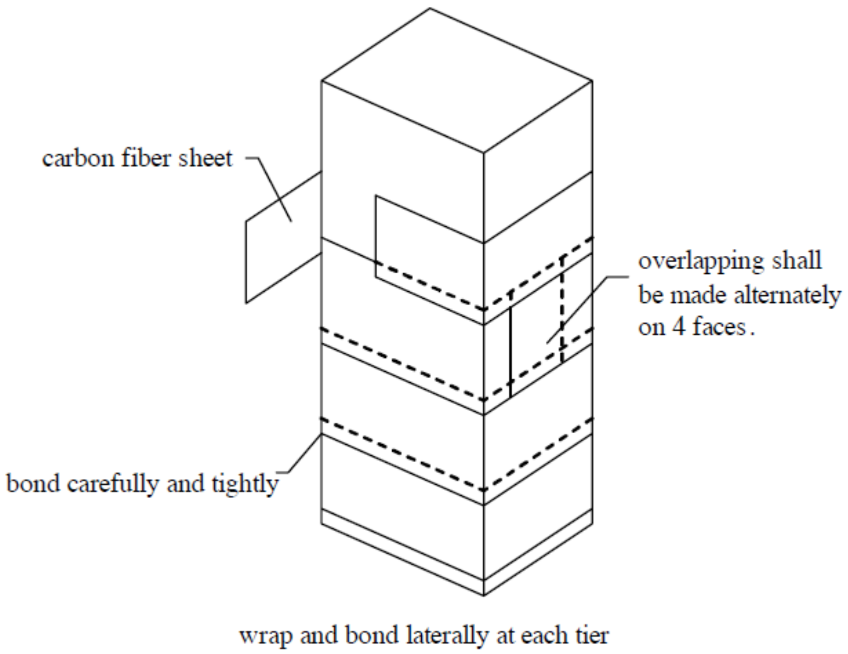


Figure TN.28 Strengthening with carbon fiber sheet wrapping

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Carbon fiber sheet wrapping

Test Work of CNCRP in 2012

Test Work of CNCRP in 2012

Carbon fiber sheet wrapping

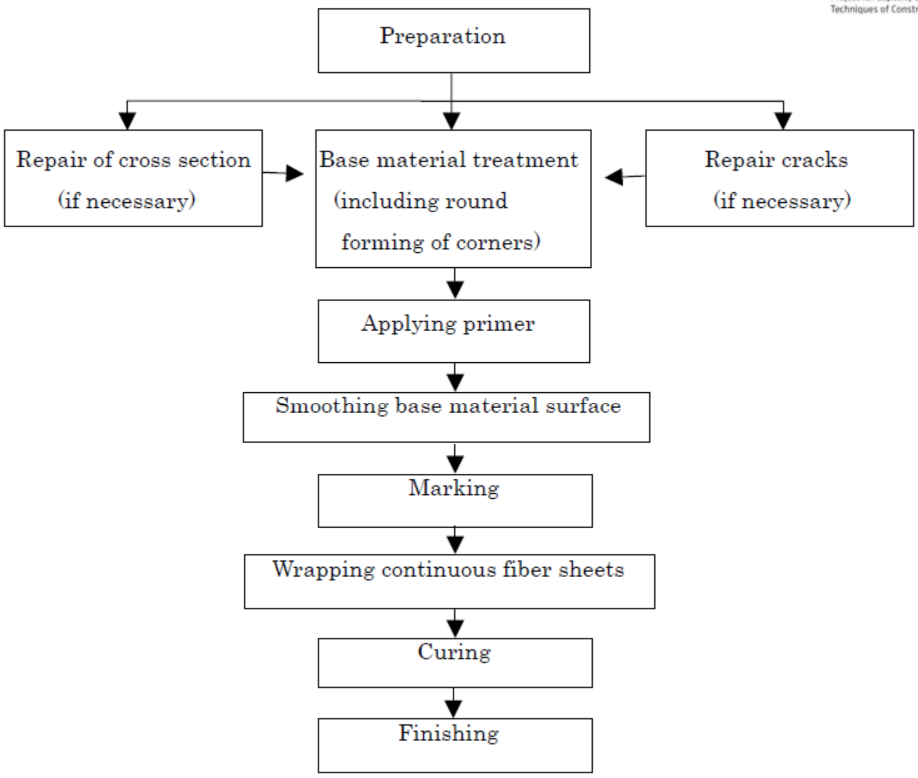


Figure 4.9-1 Flow of standard construction procedure

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Retrofitting with Beam Jacketing

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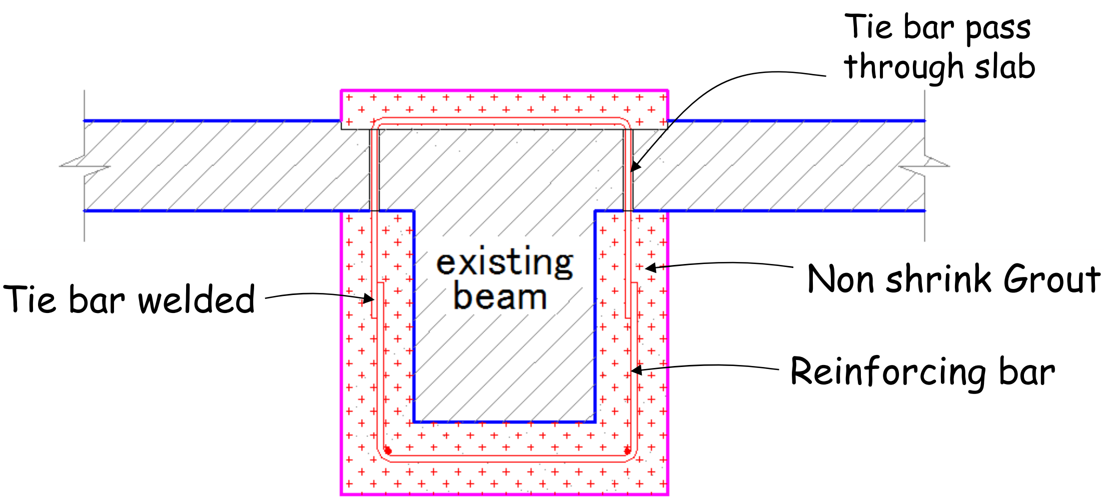

Typical Detail of Beam Jacketing

Retrofitting with Beam Jacketing

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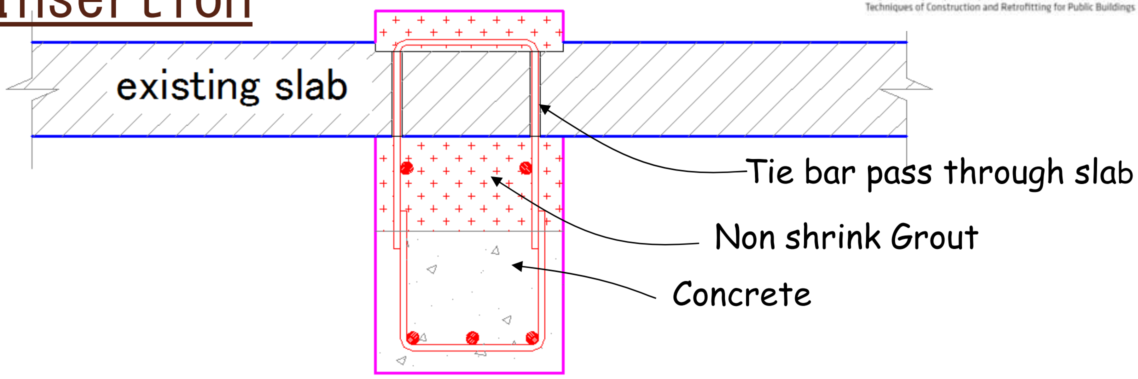
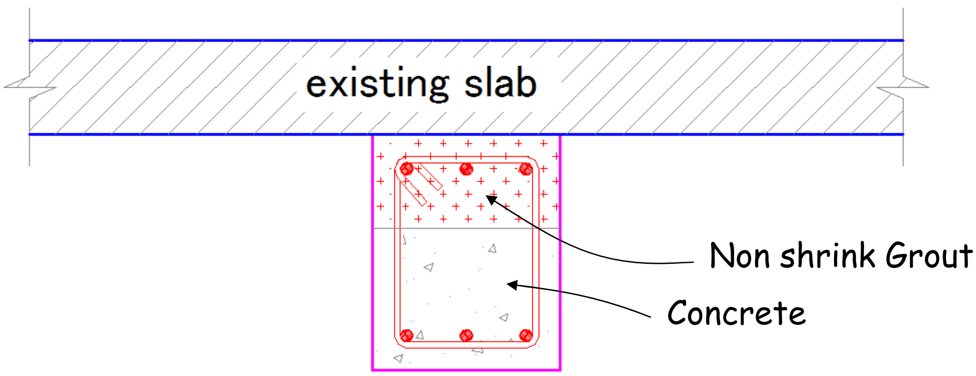
Test Work of CNCRP in 2013

Retrofitting with Beam Insertion

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings



Typical Detail of Beam Insertion (Option-1)



Typical Detail of Beam Insertion (Option-2)

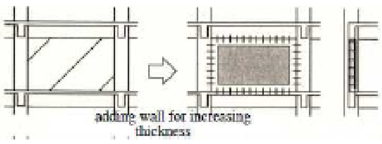
Retrofitting with Beam Insertion

Beam is inserted below existing slab

Test Work of CNCRP in 2013

Methods of Retrofitting

3. Increasing Thickness of Existing Shear Wall

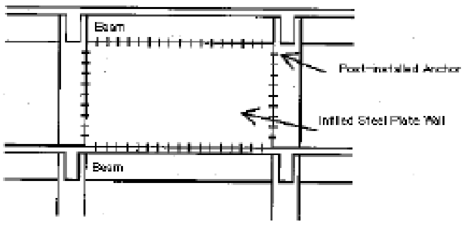


Da Vinch Ginza Building

SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Methods of Retrofitting

4. Infilling Steel Plate Wall into Open Frame

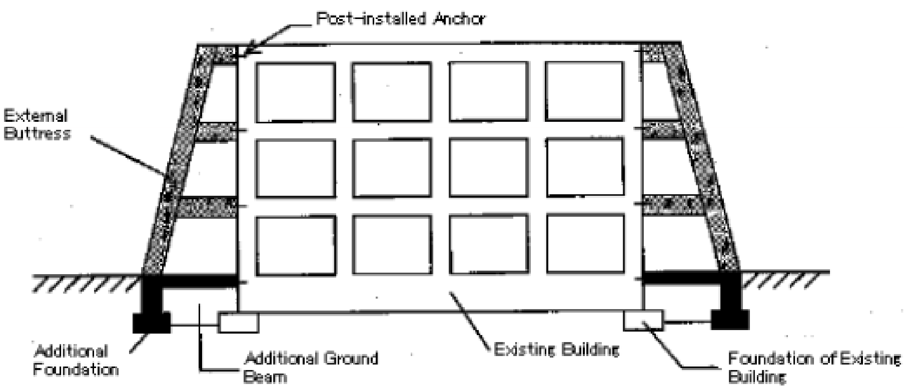


Under Construction

SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Methods of Retrofitting

7. Constructing External Buttress



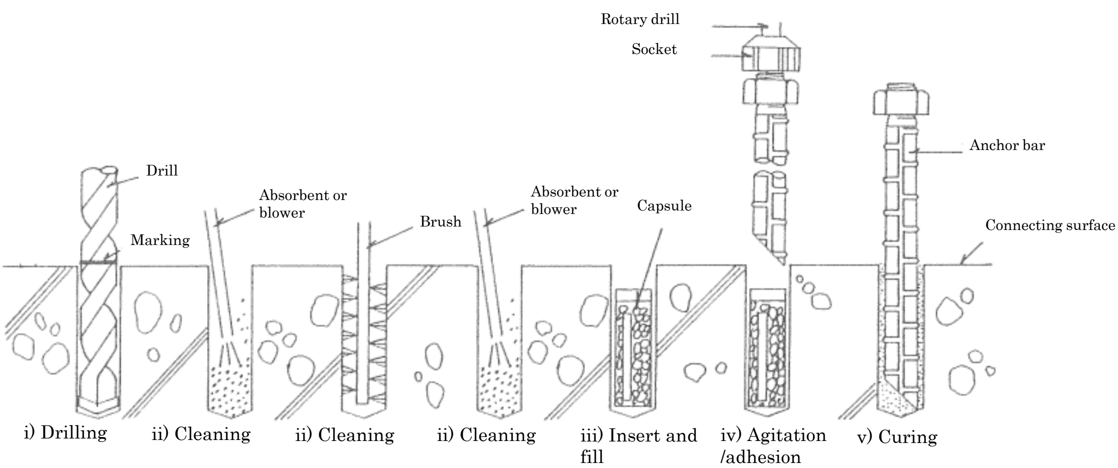
SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Base Isolation

Lead Rubber Bearing with Isolator used

Damper used

Post-Installed Anchor Work



SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Post-Installed Anchor Work

Pressurized Grouting Work

Thank you very much

SHORT TRAINING COURSE ON SEISMIC ASSESSMENT, RETROFIT DESIGN AND CONSTRUCTION OF RC BUILDING

TITLE OF LECTURE

RETROFITTING DESIGN EXAMPLE OF A REAL STRUCTURE

PRESENTED BY

ANUP KUMAR HALDER

SUB DIVISIONAL ENGINEER

PWDDESIGN DIVISION-V.

&

TEAM MEMBER WORKING TEAM-II

OUTLINE

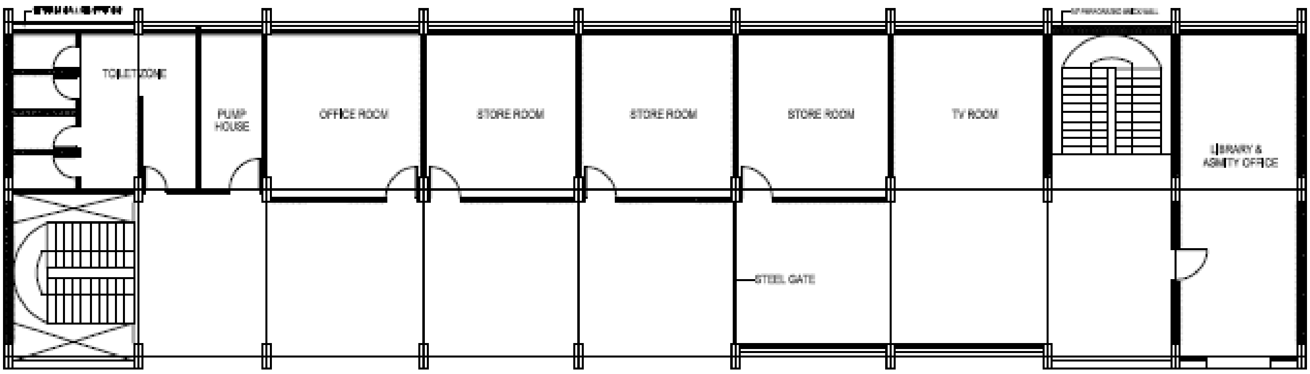
1. BUILDING VIEW/ PLAN/ LAYOUT/ELEVATION
2. INSPECTION FOR BUILDING DATA
3. ASSESSMENT IN X DIRECTION (DETAILS OF STOREY-1)
4. ASSESSMENT IN Y DIRECTION (DETAILS OF STOREY-1)
5. C, F VALUE IN X DIRECTION FLOOR WISE
6. CALCULATION OF DEMAND
7. COLUMN JACKETING
8. WING WALL
9. SHEAR WALL
10. CHECK FOR PERFORMANCE OF SW
11. CARBON FIBRE WRAPING
12. STEEL BRACING
13. SELECTION OF METHOD

BUILDING VIEW

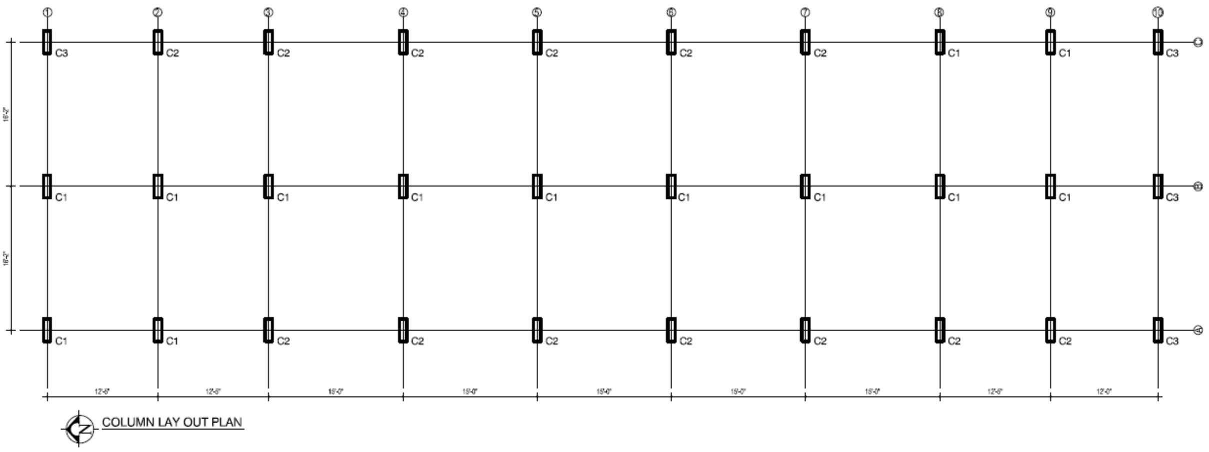
ANNEX BUILDING OF MULTISTORIED GARAGE CUM OFFICE BHABAN.

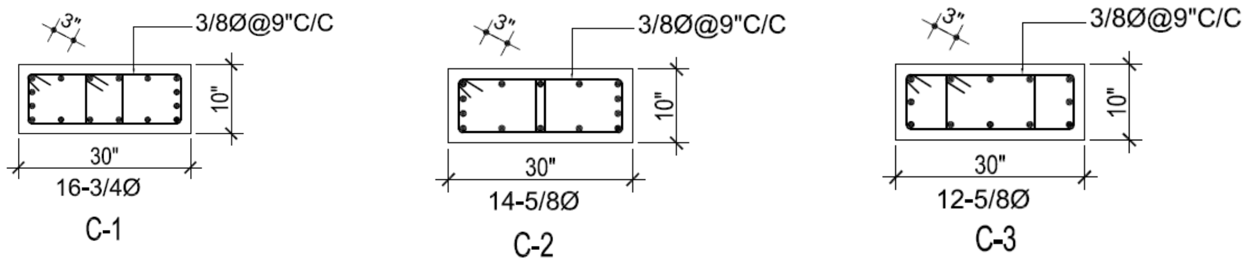
BUILDING PLAN

ANNEX BUILDING OF MULTISTORIED GARAGE CUM OFFICE BHABAN

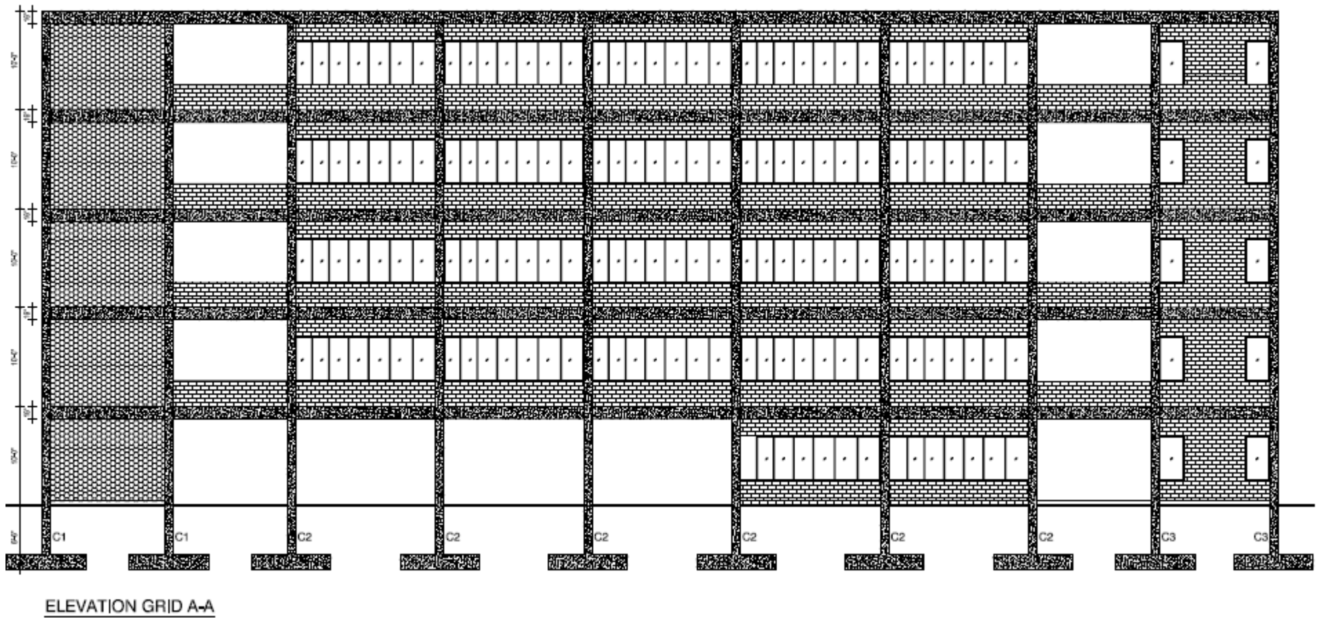


BUILDING LAYOUT
ANNEX BUILDING OF MULTISTORIED GARAGE CUM OFFICE BHABAN





ELEVATION GRID A-A
ANNEX BUILDING OF MULTISTORIED GARAGE CUM OFFICE BHABAN



INSPECTION

ANNEX BUILDING OF MULTISTORIED GARAGE CUM OFFICE BHABAN

BUILDING DATA

NAME	ANNEX BUILDING OF MULTISTORIED GARAGE CUM OFFICE BHABAN.
BUILDING USE	OFFICE
STRUCTURE TYPE	R.C.C FRAMED STRUCTURE
YEAR OF CONSTRUCTION	1985
CONCRETE f’c	9.2 Mpa (DESIGN f’c=13.7Mpa)
REBAR fy	275 Mpa
TOTAL STOREY	5(FIVE)
FLOOR AREA	377.38 Sqm
FOUNDATION TYPE	SHALLOW /DEPTH 4’-6” FROM EGL
BEARING CAP	1.00 TSF

BUILDING DATA cont.....

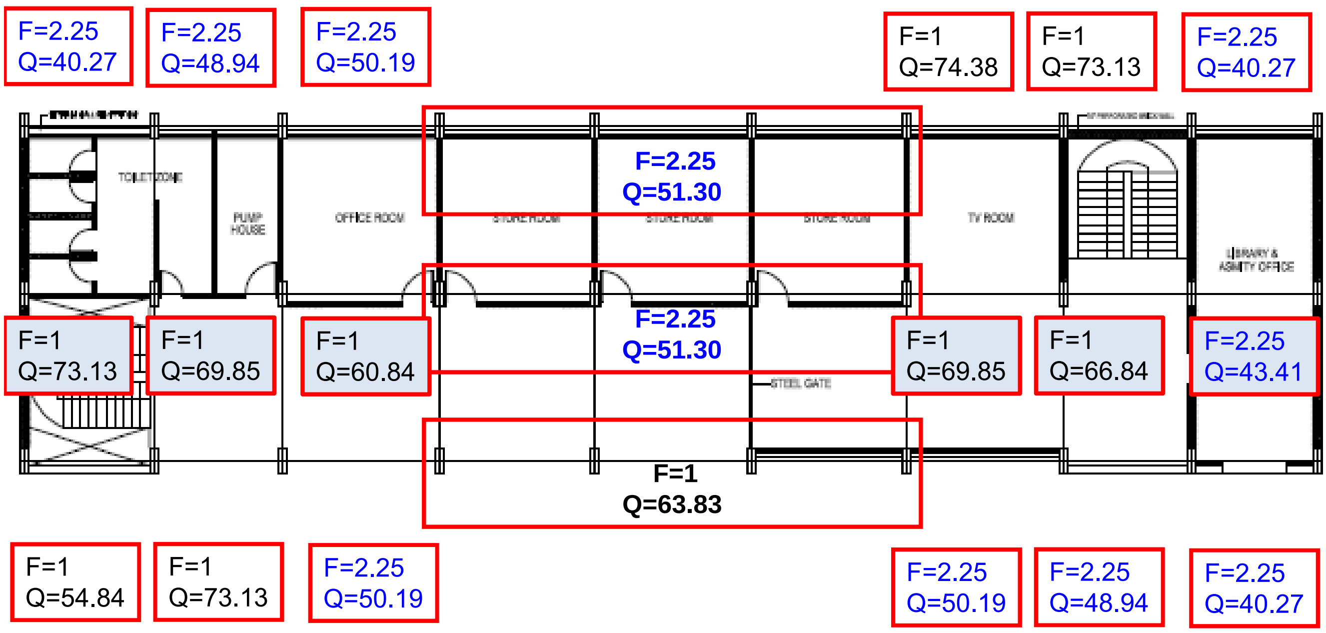
Material Properties:	
f'c(N/mm2)	9.20
σy(N/mm2)	275.00
σwy(N/mm2)	275.00

Unit Area weight(Roof):		
1. Live Load	0.30	kN/Sqm
2. Brick Wall	0.00	kN/Sqm
3. Slab weight & Floor Finish	3.85	kN/Sqm
5. SW(Column+Beam)	2.15	kN/Sqm
w =	6.3	kN/Sqm

Unit Area weight(Typical Floor):		
1. Live Load	0.80	kN/Sqm
2. Brick Wall	4.00	kN/Sqm
3. Slab weight & Floor Finish	3.85	kN/Sqm
5. SW(Column+Beam)	2.15	kN/Sqm
w =	10.8	kN/Sqm

FLOOR AREA	377.38	Sqm

BUILDING ASSESSMENT (X DIRECTION STORY 1)



F=2.25

QT=(51.30*8)+(40.27*3)+(48.94*2)+(50.19)*3+43.41=822.89 KN

C=822.89/18692.2=0.04

E_O CALCULATION (X DIR STORY 1)

STRENGTH DOMINANT STRUCTURE

Σwi= 18692.2 kN

Direction	Story	GN	Q	C	ΣQ	C1	F	E0-1	E0-2	Ctu	
X	1	1	0.0	0.00	0.00	0.000	0.80				
		2	874.3	0.05	1466.96	0.078	1.00		0.078	0.08	
		3	0.0	0.00	0.00	0.000	1.10				
		4	0.0	0.00	0.00	0.000	1.20				
		5	0.0	0.00	0.00	0.000	1.27				
		6	0.0	0.00	0.00	0.000	1.40				
		7	0.0	0.00	0.00	0.000	1.50				
		8	0.0	0.00	0.00	0.000	1.75				
		9	0.0	0.00	0.00	0.000	2.00				
		10	823.1	0.04	823.10	0.044	2.25		0.099	0.04	
		11	0.0	0.00	0.00	0.000	2.60				
		12	0.0	0.00	0.00	0.000	3.00				
		13	0.0	0.00	0.00	0.000	3.20				
		ΣQ	1697.4		MAX_E0	0.044	2.25		0.099		

DUCTILITY DOMINANT STRUCTURE

$$E_O = \sqrt{(1*0.05)^2+(2.25*0.04)^2} = 0.11$$

ASSESSMENT SUMMARY (X DIRECTION)

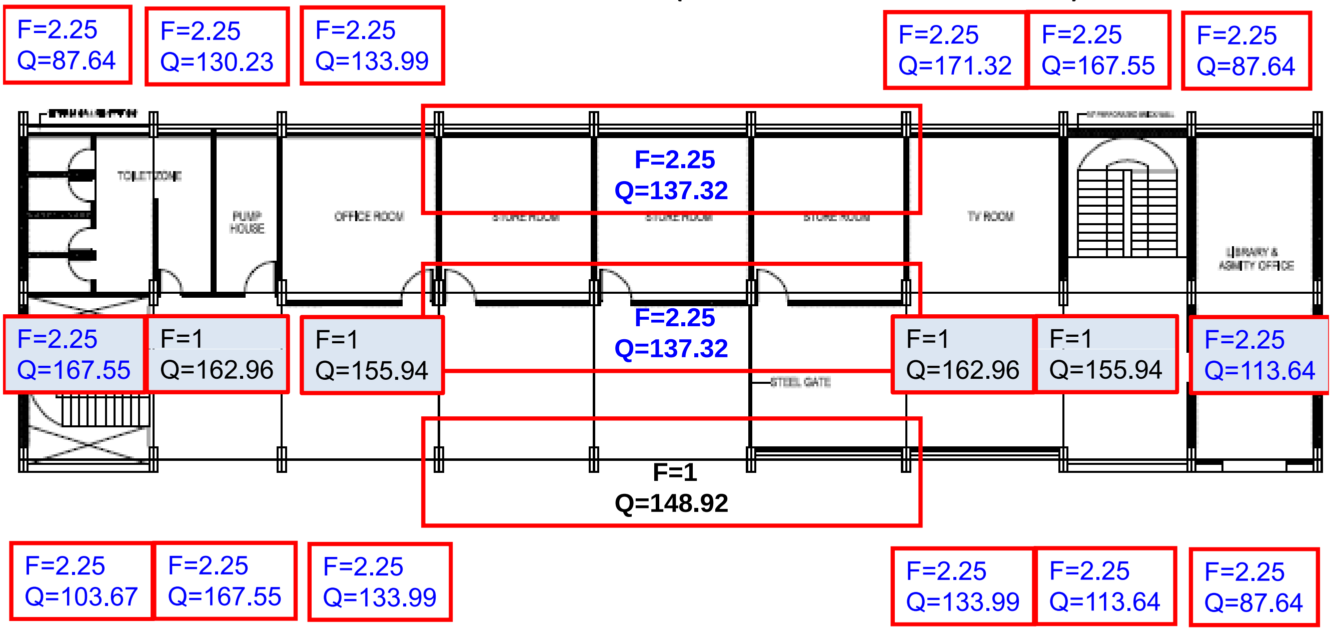
Direction	Story	C	F	Failure Mode	E_O	T	S_D	I_S	$C_{TU} \cdot S_D$	Result	Adoption	Eq
X	5											5
		0.339	2.25		0.458			0.458	0.203	OK		5
		0.478	1.00		0.540	1.000	1.000	0.540	0.203	OK		4
		0.339	2.25									
		0.722	1.00		[0.433]			0.433	[0.433]	OK		5
	4											5
		0.147	2.25		[illegible]			[illegible]	[illegible]	[illegible]		5
		0.200	1.00		0.258	1.000	1.000	[illegible]	[illegible]	[illegible]		4
		0.147	2.25									
		0.307	1.00		[0.155]			[illegible]	[illegible]	[illegible]		5
	3											5
		0.101	2.25		0.170			0.170	0.076	NG		5
		0.133	1.00		0.197	1.000	1.000	0.197	0.076	NG		4
		0.101	2.25									
		0.206	1.00		[0.154]			0.154	[0.154]	NG		5
	2											5
		0.080	2.25		0.155			0.155	0.069	NG		5
		0.095	1.00		[illegible]	[illegible]	[illegible]	0.110	0.069	NG		4
		0.080	2.25									
		0.153	1.00		[0.131]			0.131	[0.131]	NG		5
	1											5
		0.044	2.25		0.099			0.099	0.044	NG		5
		0.047	1.00		0.110	1.000	1.000	0.110	0.044	NG		4
		0.044	2.25									
		0.078	1.00		[0.078]			0.078	[0.078]	NG		5

DUCTILITY DOMINANT

STRENGTH DOMINANT

I_S CONSIDERED FOR RETROFITTING

BUILDING ASSESSMENT (Y DIRECTION STORY 1)



F=2.25

QT=(148.92*4)+(155.94*2)+(162.96*2)=1233.48 KN

C=1233.48/18692.2=0.06

CALCULATION OF Eo (STRENGTH DOMINENET STRUCTURE)

STRENGTH DOMINANT STRUCTURE

N+1/N+i= 1.000

Σwi= 18692.2 kN

Direction	Story	GN	Q	C	ΣQ	C1	F	E0-1	E0-2	Ctu	
		1	0.0	0.00	0.00	0.000	0.80				
		2	1233.5	0.07	3320.48	0.178	1.00		0.178	0.18	
		3	0.0	0.00	0.00	0.000	1.10				
		4	0.0	0.00	0.00	0.000	1.20				
		5	0.0	0.00	0.00	0.000	1.27				
		6	0.0	0.00	0.00	0.000	1.40				
Y	1	7	0.0	0.00	0.00	0.000	1.50				
		8	0.0	0.00	0.00	0.000	1.75				
		9	0.0	0.00	0.00	0.000	2.00				
		10	2898.6	0.16	2898.61	0.155	2.25		0.349	0.16	
		11	0.0	0.00	0.00	0.000	2.60				
		12	0.0	0.00	0.00	0.000	3.00				
		13	0.0	0.00	0.00	0.000	3.20				
		ΣQ	4132.1	MAX_E0		0.155	2.25		0.349		

DUCTILITY DOMINANT STRUCTURE

$$E_O = \sqrt{(1*0.07)^2+(2.25*0.16)^2}=0.36$$

ASSESSMENT SUMMARY (Y DIRECTRION)

Seismic demand index							Iso=	0.30	$C_{TU} \cdot S_D$=	0.15		
Direction	Story	C	F	Failure Mode	Eo	T	S_D	Is	$C_{TU} \cdot S_D$	Result	Adoption	Eq
												5
		1.789	1.50		1.610			1.610	1.073	OK		5
	5	0.646	1.50		1.649	1.000	1.000	1.649	0.686	OK		4
		1.143	2.25									
												5
												5
		0.746	1.20		0.596			0.596	0.497	OK		5
	4	0.391	1.20		0.651	1.000	1.000	0.651	0.254	OK		4
		0.381	2.25									
												5
												5
		0.515	1.20		0.463			0.463	0.386	OK		5
Y	3	0.268	1.20		0.507	1.000	1.000	0.507	0.198	OK		4
		0.265	2.25									
												5
												5
		0.214	2.00		0.367			0.367	0.184	OK		5
	2	0.189	1.00		0.355	1.000	1.000	0.355	0.184	OK		4
		0.214	2.00									
		0.343	1.00		[0.294]			0.294	[0.294]	NG		5
												5
		0.155	2.25		0.349			0.349	0.155	OK		5
	1	0.066	1.00		0.355	1.000	1.000	0.355	0.155	OK		4
		0.155	2.25									
		0.178	1.00		[0.178]			0.178	[0.178]	NG		5

ASSESSMENT IN Y DIR STORY 1

Seismic demand index							Iso=	0.30	$C_{TU} \cdot S_D$=	0.15		
												5
		0.155	2.25		0.349			0.349	0.155	OK		5
Y	1	0.066	1.00		0.355	1.000	1.000	0.355	0.155	OK		4
		0.155	2.25									
		0.178	1.00		[0.178]			0.178	[0.178]	NG		5

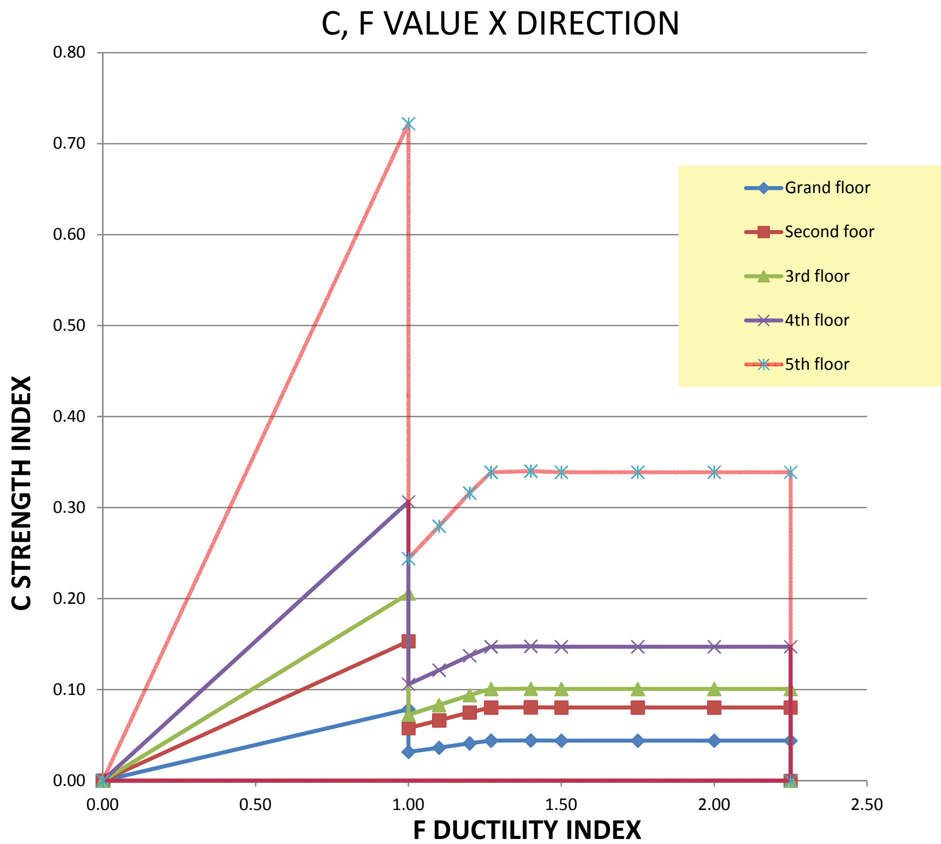
N+1/N+i= 1.000

Σwi= 18692.2 kN

Direction	Story	GN	Q	C	ΣQ	C1	F	E0-1	E0-2	Ctu	
		1	0.0	0.00	0.00	0.000	0.80				
		2	1233.5	0.07	3320.48	0.178	1.00		0.178	0.18	
		3	0.0	0.00	0.00	0.000	1.10				
		4	0.0	0.00	0.00	0.000	1.20				
		5	0.0	0.00							
		6	0.0	0.00							
Y	1	7	0.0	0.00							
		8	0.0	0.00							
		9	0.0	0.00	0.00	0.000	2.00				
		10	2898.6	0.16	2898.61	0.155	2.25		0.349	0.16	
		11	0.0	0.00	0.00	0.000	2.60				
		12	0.0	0.00	0.00	0.000	3.00				
		13	0.0	0.00	0.00	0.000	3.20				
		ΣQ	4132.1		MAX_E0	0.155	2.25		0.349		

DUCTILITY DOMINANT STRUCTURE

$E_O = \sqrt{(1*0.07)^2+(2.25*0.16)^2} = 0.36$



CALCULATION OF DEMAND

- $I_{SO}=E_OxS_DxT=C1xFXS_DxT=C1=\sum Q1/W$
- $I_{SX}=E_OxS_DxT=C2xFXS_DxT=C2=\sum Q2/W$
- CONSIDERING NO CHANGE IN THE SYSTEM WITH (FXS_DxT)
- $I_{SO}-I_{SX}=\sum Q1/W-\sum Q2/W$
- $(I_{SO}-I_{SX})XW=\sum Q1-\sum Q2$=REQUIRED SHEAR CAPACITY
- $I_{SO}=0.3$
- $I_{SX}=0.078$
- $I_{SY}=0.178$
- W=18692.2 KN
- SHEAR REQUIREMENT IN X=(0.3-0.078)X18692.2=4150KN
- SHEAR REQUIREMENT IN Y=(0.3-0.178)X18692.2=2280KN

COLUMN JACKETING

When $0.4b \cdot D \cdot F_{c1} \geq N \geq 0$,

$$M_u = a_t \cdot \sigma_y \cdot g + a_{t2} \cdot \sigma_{y2} \cdot g_2 + 0.5 \cdot N \cdot D_2 \cdot \left(1 - \frac{N}{b_2 \cdot D_2 \cdot F_{c1}}\right)$$

Old rebar

Concrete

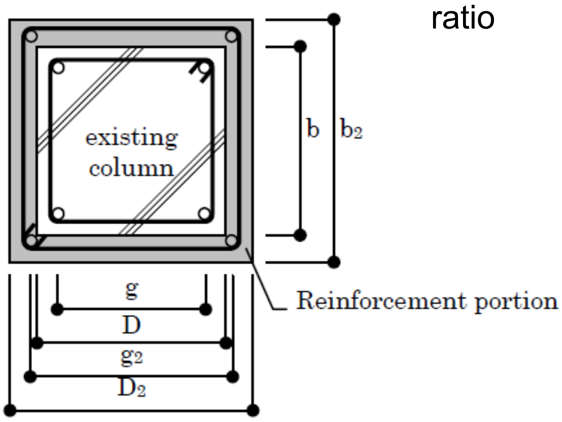
Area of tension rebar in jacketing part

$$Q_{su} = \phi \left\{ \frac{0.053 \cdot p_{t2}^{0.23} \cdot (F_{c1} + 18)}{M/(Q \cdot d_2) + 0.12} + 0.85\sqrt{p_w \cdot \sigma_{wy} + p_{w2} \cdot \sigma_{wy2}} + 0.1\frac{N}{b_2 \cdot D_2} \right\} \times 0.8 \cdot b_2 \cdot D_2$$

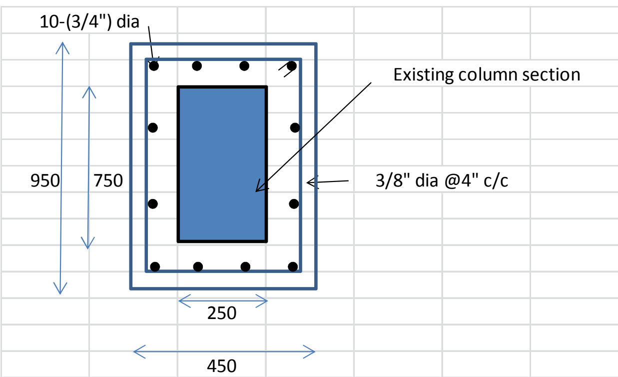
Shear reinforcement

Axial force ratio

Concrete strength, main rebar ratio, slenderness



COLUMN JACKETING cont…



- SHEAR REQUIREMENT IN X=4150KN

X-DIRECTION
Q =150KN
Qpre=70KN
Qgain=80KN
NOT GOOD

- SHEAR REQUIREMENT IN Y=2280KN

Y-DIRECTION
Q =242KN
Qpre=130KN
Qgain=112KN
Appx. 20Column

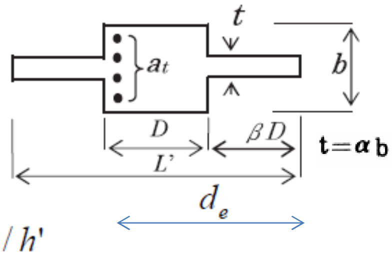
WING WALL

- CONTRIBUTION OF TENSION SIDE WING WALL IGNORED

$$M_u = (0.9+\beta)\cdot a_t \cdot \sigma_y \cdot D + 0.5N \cdot D\left\{1+2\beta - \frac{N}{\alpha_e \cdot b \cdot D \cdot F_{c1}}\left(\frac{a_t \cdot \sigma_y}{N}+1\right)^2\right\}$$

$$\alpha_e = (1+2\alpha\cdot\beta)/(1+2\beta)$$

T-SIDE



C-SIDE

$$Q_{mu} = \phi \cdot M_u / h'$$

$$Q_{su} = \phi\left\{\frac{0.053\cdot p_{te}^{0.23}\cdot(F_c+18)}{M/(Q\cdot d_e)+0.12} + 0.85\sqrt{p_{we}\cdot\sigma_{wy}} + 0.1\sigma_{oe}\right\}\cdot b_e \cdot j_e$$

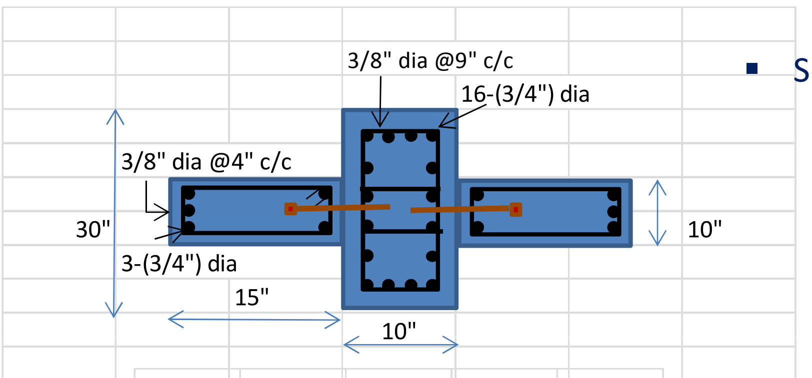
$$p_{te} = 100a_t/(b_e \cdot d_e) \qquad p_{we}\cdot\sigma_{wy} = p_w\cdot\sigma_{wy}(b/b_e) + p_{sh}\cdot\sigma_{sy}(t/b_e)$$

$$\sigma_{oe} = N/(b_e \cdot j_e)$$

LATERAL REBAR RATIO(EXISTING COL+ WING WALL)

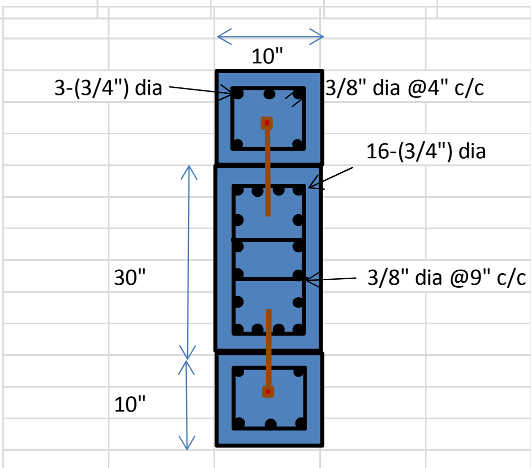
$$b_e = \alpha_e \cdot b$$

WING WALL cont…



- SHEAR REQUIREMENT IN X=4150KN

X-DIRECTION
Q =300KN
<u>Qpre=70KN</u>
Qgain=230KN
Appx. 18Column



- SHEAR REQUIREMENT IN Y=2280KN

Y-DIRECTION
Q =270KN
<u>Qpre=130KN</u>
Qgain=140KN
Appx. 17Column

SHEAR WALL CALCULATION

SHEAR STRENGTH OF SW

$$ {}_{W}Q_{su} = \min\left\{ {}_{W}Q'_{su} + 2\cdot\alpha\cdot Q_c \,,\; Q_j + {}_{p}Q_c + \alpha\cdot Q_c \right\} $$

- Shear force of column
- Direct shear strength at top of col
- Shear strength of infill panel
- Shear connector

$$ {}_{W}Q'_{su} = \max\left(p_w \cdot {}_{W}\sigma_y \,, F_{cw}/20 + 0.5\,p_w \cdot {}_{W}\sigma_y\right)\cdot t_W \cdot l' $$

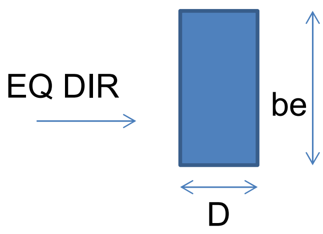
- Wall reinforcement ratio and yield strength
- Wall thickness & clear span

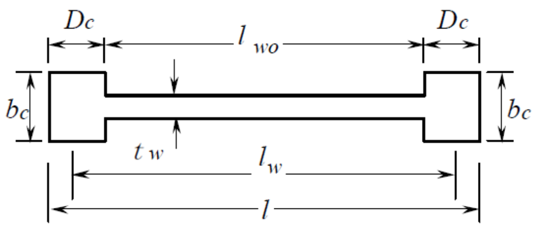
SHEAR STRENGTH COLUMN

$$ {}_{p}Q_c = K_{\min}\cdot\tau_o\cdot b_e\cdot D $$

Q_j = Sum of the shear strengths of connectors underneath the beam.

SHEAR WALL CALCULATION cont..





$$ \tau_o = 0.66F_{c1} \quad \text{in case} \quad 0.66 < \sigma $$

$$ \sigma_o = N/(b_e \bullet D). $$

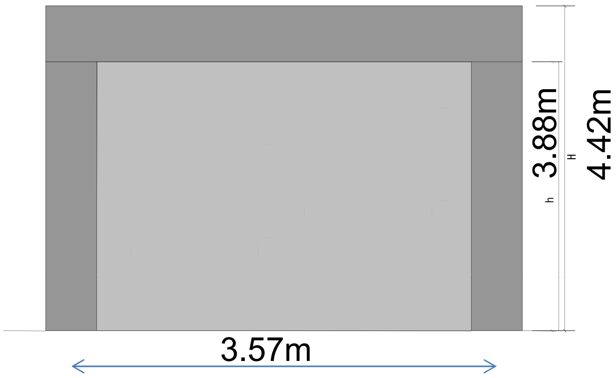
$$ \sigma = p_g\cdot\sigma_y + \sigma_o $$

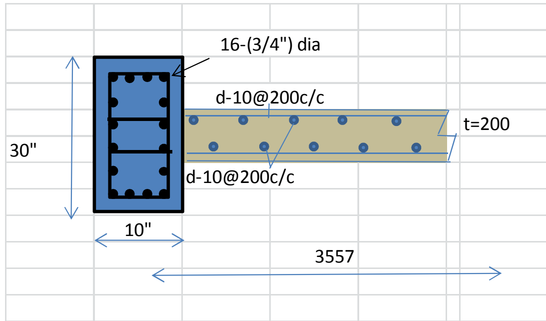
$$ p_g = a_g/b_e\cdot D $$

FLEXURAL STRENGTH OF SW

$$ {}_{W}M_u = a_t\cdot\sigma_{sy}\cdot l_W + 0.5\sum(a_{wy}\cdot\sigma_{wy})\cdot l_W + 0.5N\cdot l_W $$

SHEAR WALL CALCULATION cont..





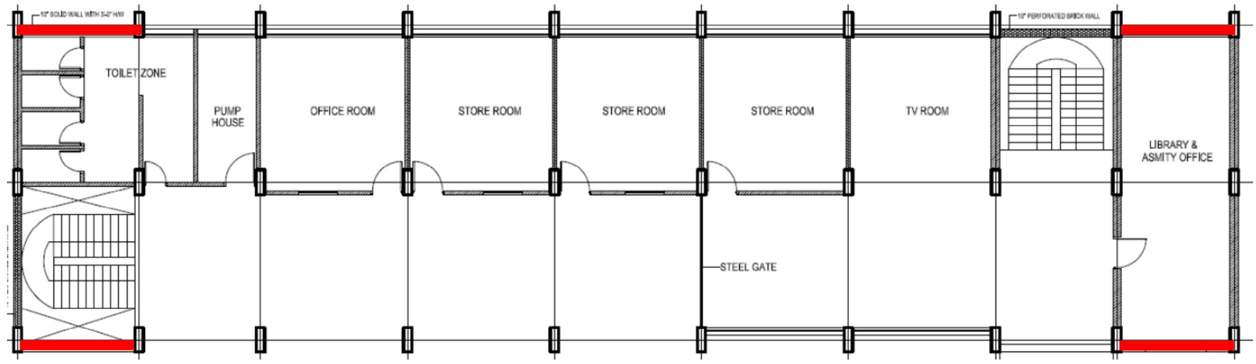
$_{W}Q_{su}$ =1400KN (X-DIRECTION)

$_{W}Q_{su}$ =1500KN (Y-DIRECTION)

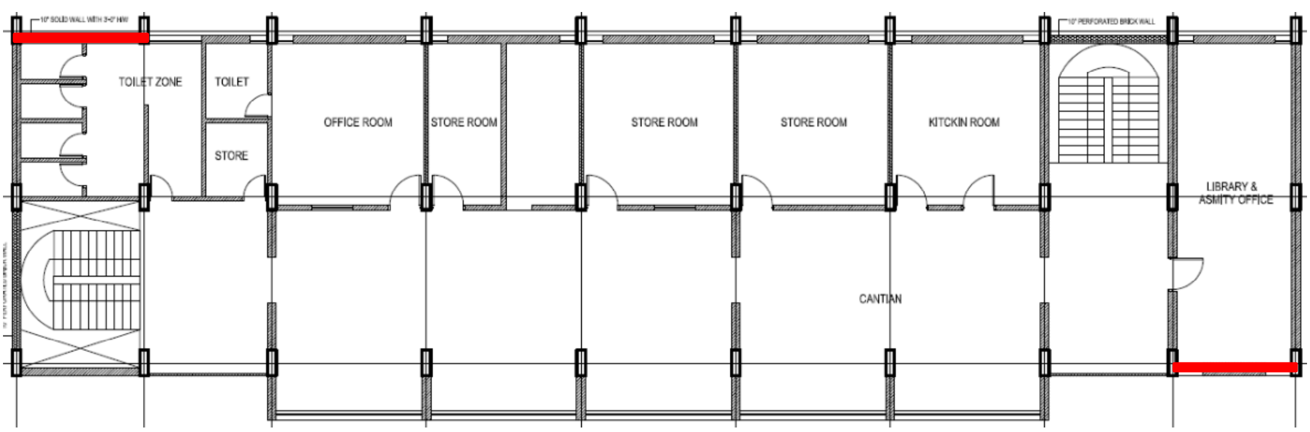
CALCULATION FOR SW(X- DIR)

Direction	Story	C	F	Eo	T	S_D	Is	$C_{TU} \cdot S_D$	Result	Adoption	Eq	WT (KN)	n+1/(n+i)	SW Cap (KN)	Req SW No=(Is0-Is)XWt/SW cap	No of SW
	5										5					
		0.339	2.25	0.458			0.458	0.203	OK		5					
		0.478	1.00		1.000	1.000			OK		4					
		0.339	2.25	0.540			0.540	0.203								
		0.722	1.00	[0.433]			0.433	[0.433]	OK		5	2377.494	0.6	840	-0.4	Not Required
	4										5					
		0.147	2.25	0.221			0.221	0.098	NG		5					
		0.200	1.00		1.000	1.000										
		0.147	2.25	0.258			0.258	0.980	NG		4					
		0.307	1.00	[0.155]			0.155	[0.155]	NG		5	6453.198	0.67	933.3333333	0.6	1
	3										5					
		0.101	2.25	0.170			0.170	0.076	NG		5					
		0.133	1.00		1.000	1.000					4					
		0.101	2.25	0.197			0.197	0.076	NG							
X		0.206	1.00	[0.154]			0.154	[0.154]	NG		5	10528.902	0.75	1050	1.4	1
	2										5					
		0.080	2.25	0.155			0.155	0.069	NG		5					
		0.095	1.00		1.000	1.000										
		0.080	2.25	0.110			0.110	0.069	NG		4					
		0.153	1.00	[0.131]			0.131	[0.131]	NG		5	14604.606	0.857142857	1200	1.9	2
	1										5					
		0.044	2.25	0.099			0.099	0.044	NG		5					
		0.180	1.00		1.000	1.000										
		0.044	2.25	0.110			0.110	0.044	NG		4					
		0.078	1.00	[0.078]			0.078	[0.078]	NG		5	18680.31	1	1400	2.7	3

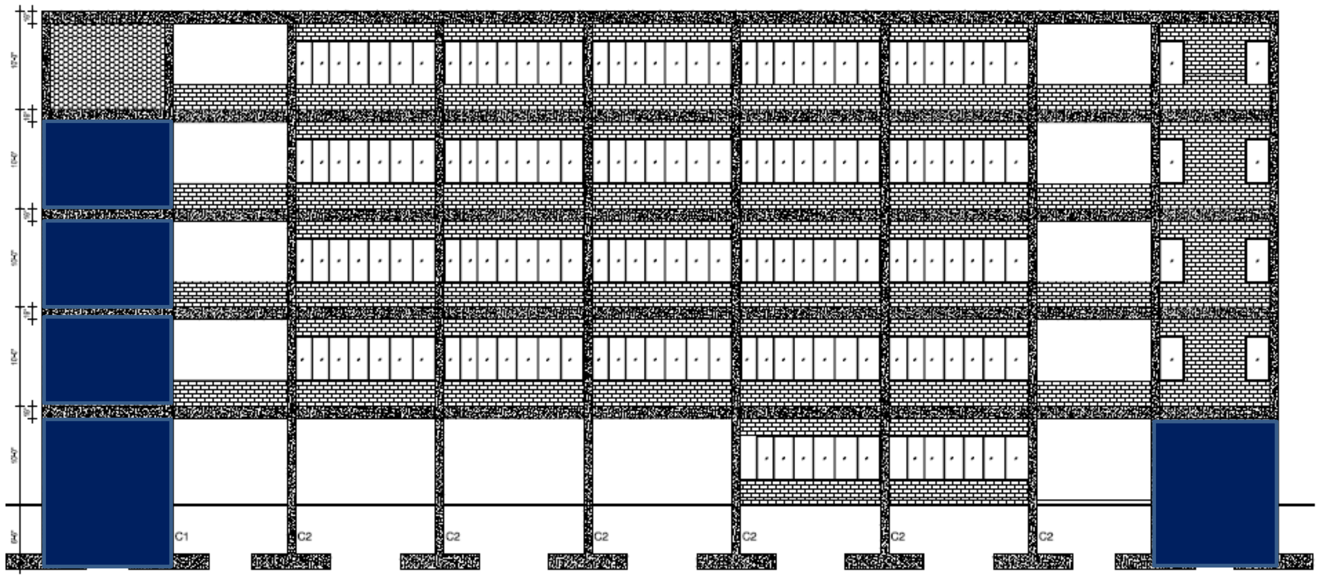
ADDING SW IN X-DIR (GROUND FLOOR)



ADDING SW IN X-DIR (1ST FLOOR,2ND ,3RD FLOOR)

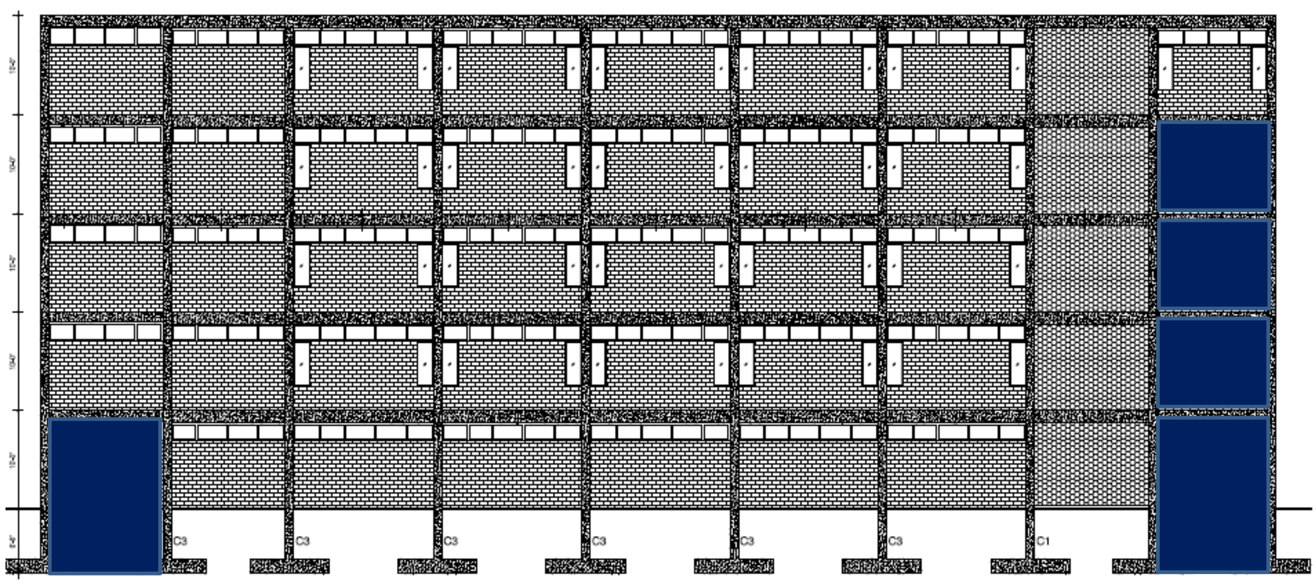


ELEVATION GRID A-A



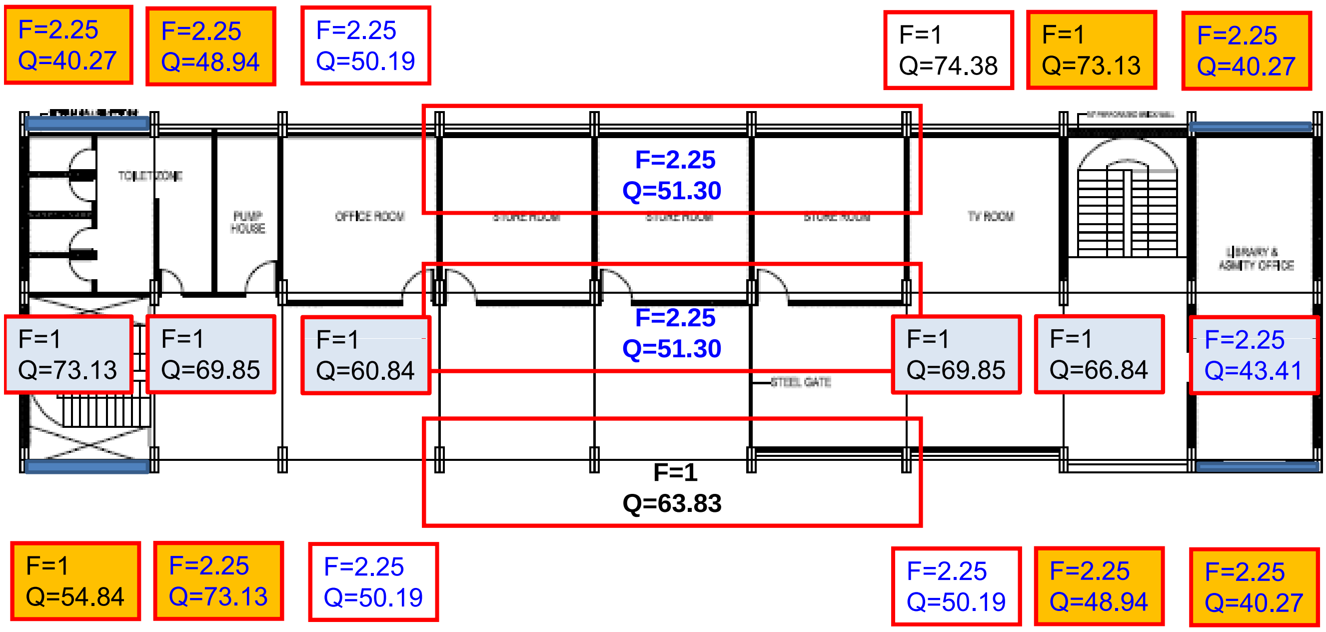
ELEVATION GRID A-A

ELEVATION GRID C-C



ELEVATION GRID C-C

BUILDING ASSESSMENT (X DIRECTION STORY 1 AFTER)



F=1 QT(-)=874.3-(54.84+73.13)+(0.72)(823.1-40.27*3-48.94*2)+4*1400=6781.5

C=6781.5/18692.2=0.36

F=2.25 QT=823.1-40.27*3-48.94*2=604.41KN

C=604.41/18692.2=0.032

E_O CALCULATION (X DIR STORY 1 AFTER INSERTION OF 4-WALL)

STRENGTH DOMINANT STRUCTURE

Σwi= 18692.2 kN

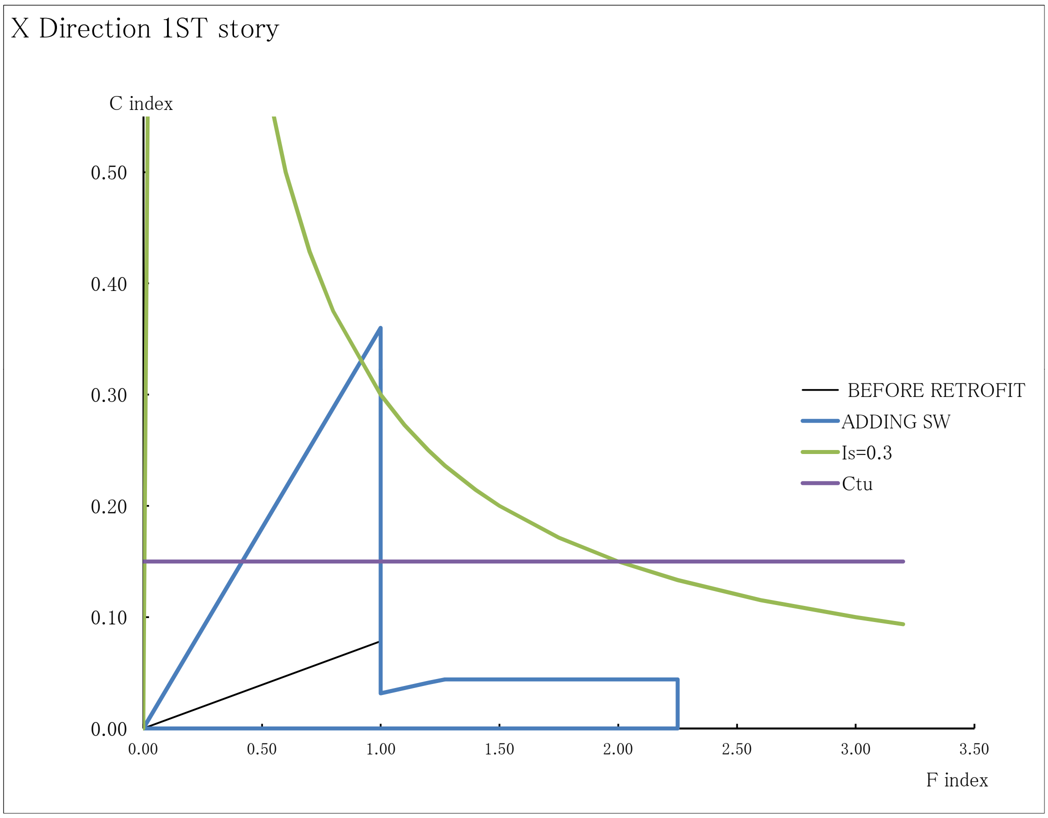
Direction	Story	GN	Q	C	ΣQ	C1	F	E0-1	E0-2	Ctu	
X	1	1	0.0	0.00	0.00	0.000	0.80				
		2	6346.33	0.33	6781.50	0.36	1.00		0.36	0.36	
		3	0.0	0.00	0.00	0.000	1.10				
		4	0.0	0.00	0.00	0.000	1.20				
		5	0.0	0.00	0.00	0.000	1.27				
		6	0.0	0.00	0.00	0.000	1.40				
		7	0.0	0.00	0.00	0.000	1.50				
		8	0.0	0.00	0.00	0.000	1.75				
		9	0.0	0.00	0.00	0.000	2.00				
		10	604.41	0.032	604.41	0.032	2.25		0.072	0.032	
		11	0.0	0.00	0.00	0.000	2.60				
		12	0.0	0.00	0.00	0.000	3.00				
		13	0.0	0.00	0.00	0.000	3.20				
		ΣQ	1697.4		MAX_E0	0.36	1.0		0.36		

DUCTILITY DOMINANT STRUCTURE

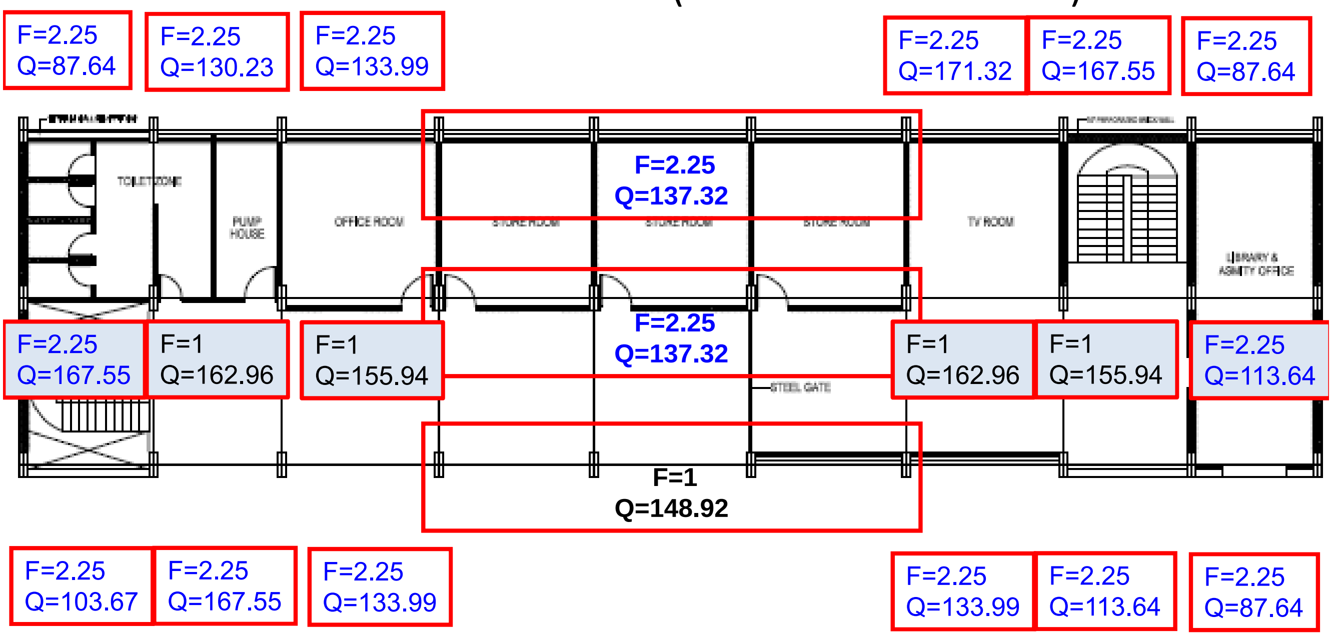
$$E_O = \sqrt{(1*0.36)^2+(2.25*0.032)^2} = 0.36$$

PERFORMANCE OF SHEAR WALL

C, F VALUE AFTER ADDING SW IN X DIR 1ST STOREY



BUILDING ASSESSMENT (Y DIRECTION STORY 1)



F=2.25

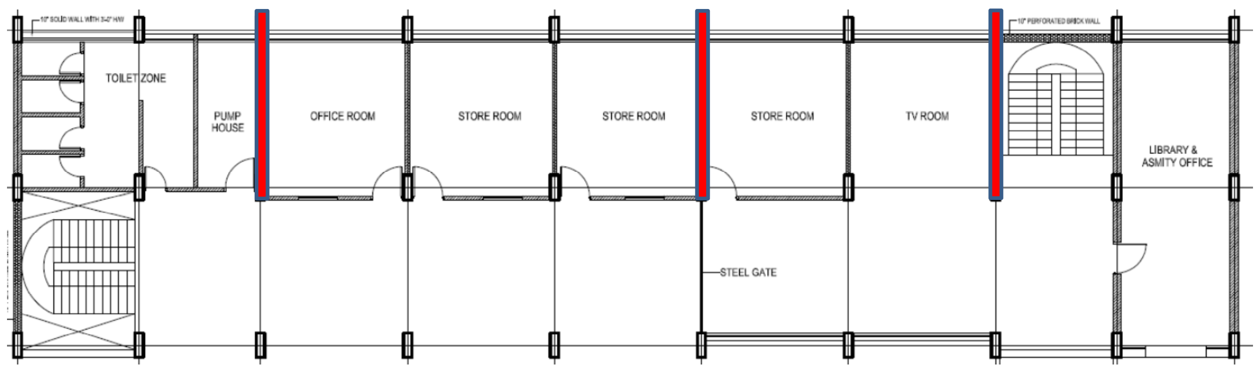
QT=(148.92*4)+(155.94*2)+(162.96*2)=1233.48 KN

C=1233.48/18692.2=0.06

CALCULATION FOR SW(Y-DIR)

Direction	Story	C	F	Eo	T	S_D	Is	$C_{TU}\cdot S_D$	Result	Adoption	Eq	WT (KN)	n+1/(n+i)	SW Cap (KN)	Req SW No=(Is0-Is)XWt/SW cap	No of SW
											5					
		1.789	1.50	1.610			1.610	1.073	OK		5					
	5	0.646	1.50	1.649	1.000	1.000	1.649	0.686	OK		4					
		1.143	2.25													
											5	2377.494	0.6	900	-3.2	Not Required
											5					
		0.746	1.20	0.596			0.596	0.497	OK		5					
	4	0.391	1.20	0.570	1.000	1.000	0.570	0.254	OK		4					
		0.381	2.25													
											5	6453.198	0.67	1000	-1.7	Not Required
											5					
		0.515	1.20	0.463			0.463	0.386	OK		5					
Y	3	0.268	1.20	0.507	1.000	1.000	0.507	0.198	OK		4					
		0.265	2.25													
											5	10528.902	0.75	1125	-1.6	Not Required
											5					
		0.214	2.00	0.367			0.367	0.184	OK		5					
	2	0.189	1.00	0.355	1.000	1.000	0.355	0.184	OK		4					
		0.214	2.00													
		0.343	1.00	[0.294]			0.294	[0.294]	NG		5	14604.606	0.857142857	1285.714286	-0.5	Not Required
											5					
		0.155	2.25	0.349			0.349	0.155	OK		5					
	1	0.066	1.00	0.355	1.000	1.000	0.355	0.155	OK		4					
		0.155	2.25													
		0.178	1.00	[0.178]			0.178	[0.178]	NG		5	18680.31	1	1500	1.6	2

ADDING SW IN Y-DIR (GROUND FLOOR)



CARBON FIBER

$$Q_{su} = \left\{ \frac{0.053 \cdot p_t^{0.23} \cdot (F_{c1} + 18)}{M/(Q \cdot d) + 0.12} + 0.85\sqrt{p_w \cdot \sigma_{wy} + p_{wf} \cdot \sigma_{fd}} + 0.1\sigma_o \right\} \cdot b \cdot j$$

$p_w \cdot \sigma_{wy} + p_{wf} \cdot \sigma_{fd}$ shall be not more than 9.8 N/mm^2.

carbon fibar		
roll	3	
thickness	0.167	mm
tensile strength	3430	N/mm2
Young's modulus	230000	N/mm2
σ fd	1610	N/mm2
Pwf	0.00401	

p_{wf} = Shear reinforcement ratio of carbon fiber sheet (decimal).

E_{fd} = Young's modulus of carbon fiber sheet

ε_{fd} = Effective strain of carbon fiber sheet at shear failure.

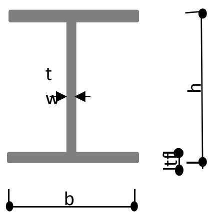
$\sigma_{fd} = \min\{E_{fd} \cdot \varepsilon_{fd}, (2/3) \cdot \sigma_f\}$, tensile strength of carbon fiber sheet for shear

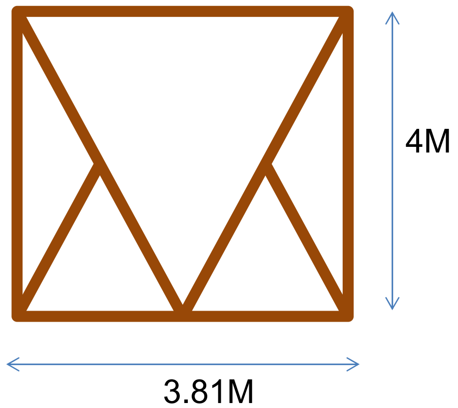
σ_f = Specified tensile strength of carbon fiber sheet.

X DIRECTION:
Qsu=105 KN; F=3.2

Y DIRECTION:
Qsu=256 KN; F=1

STEEL FRAME

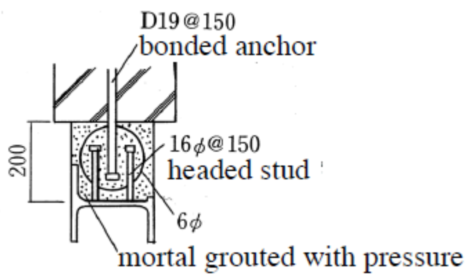




H-250X250X9X14
SS400

X-DIRECTION
Q=2079KN

Y-DIRECTION
Q=1807KN



SELECTION OF METHOD

CHOICE OF METHOD SHOULD CONSIDER
1.USE OF LOCAL AVAILABLE MATERIAL
2.ECONOMY
3.CONSTRUCTION TIME
4.EASE OF CONSTRUCTION
5.QUALITY CONTROL
6.RELABILITY OF METHOD BASED ON TEST DATA
7.ARCHITECTURAL SIMPLICITY & COHERENCE
8.UNINTERREPTED USE DURING RETROFITTING
9.LESS DISTURB THE OCCUPANT
10.MINIMUN MODIFICATION IN PURPOSE OR USE.

THANK YOU

Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings (CNCRP)

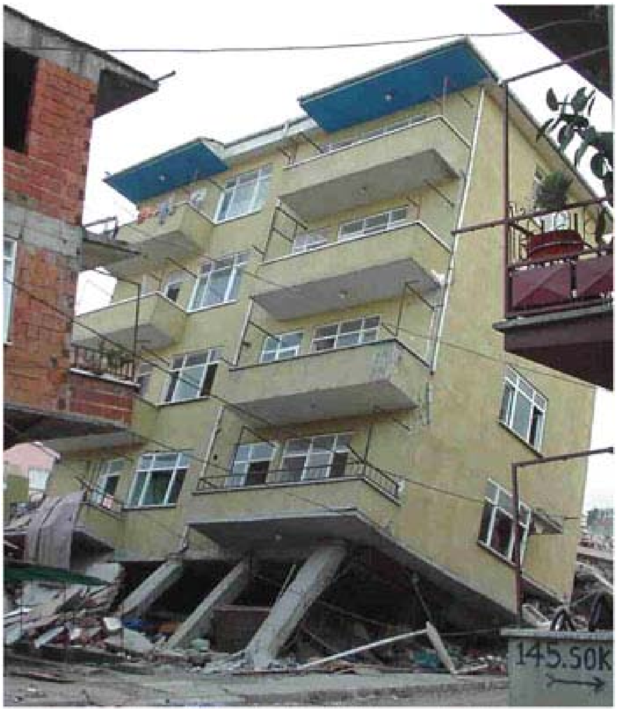
Presentation
on
Pushover Analysis for Retrofitting Design
by

Moniruzzaman Moni
Member, Working Team-2

Introduction

- Nonlinear static analysis or pushover analysis has been developed over the past thirty years
- It is the preferred analysis procedure for design and seismic performance evaluation

Introduction



Introduction

Introduction

Taiwan,1999

Earthquake happens every day somewhere in the world
Earthquake causes loss of human lives and damage of infrastructures

Introduction

80-90% of houses were permanently damaged

Learning from Earthquakes - Sichuan, 2008

Introduction

- Nonlinear Static Procedures (NSP) shall be used for analysis of buildings when linear procedures are not permitted

- The NSP shall be permitted for structures in which higher mode effects are not significant

Key Elements of the Pushover Analysis

- Nonlinear static procedure: constant gravitational loads and monotonically increasing lateral loads
- Plastic mechanisms and P-Δ effects: diplacement or arc length control
- Estimation of the target displacement: elastic or inelastic response spectrum for equivalent SDOF system

Key Elements of the Pushover Analysis

- Lateral load patterns: uniform, modal, ELF force distribution
- Capacity curve: Control node displacement vs base shear force
- Performance evaluation: global and local seismic demands with capacities of performance level

Pushover Modeling (Elements)

- Types of Elements

Truss - yielding and buckling

3D Beam - major direction flexural and shear hinging

3D Column - P-M-M interaction and shear hinging

Panel zone - Shear yielding

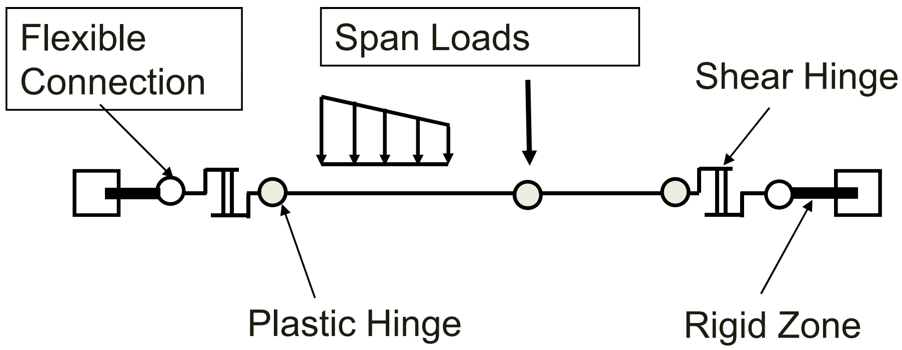
In-fill panel - Shear failure

Shear wall - P-M-Shear interaction

Spring - for foundation modeling

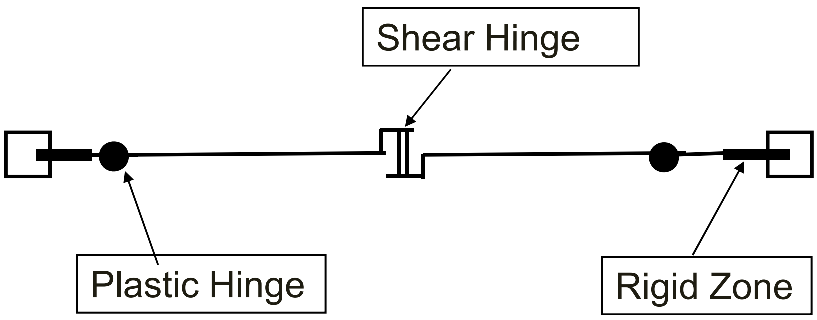
Pushover Modeling (Beam Element)

Three dimensional Beam Element

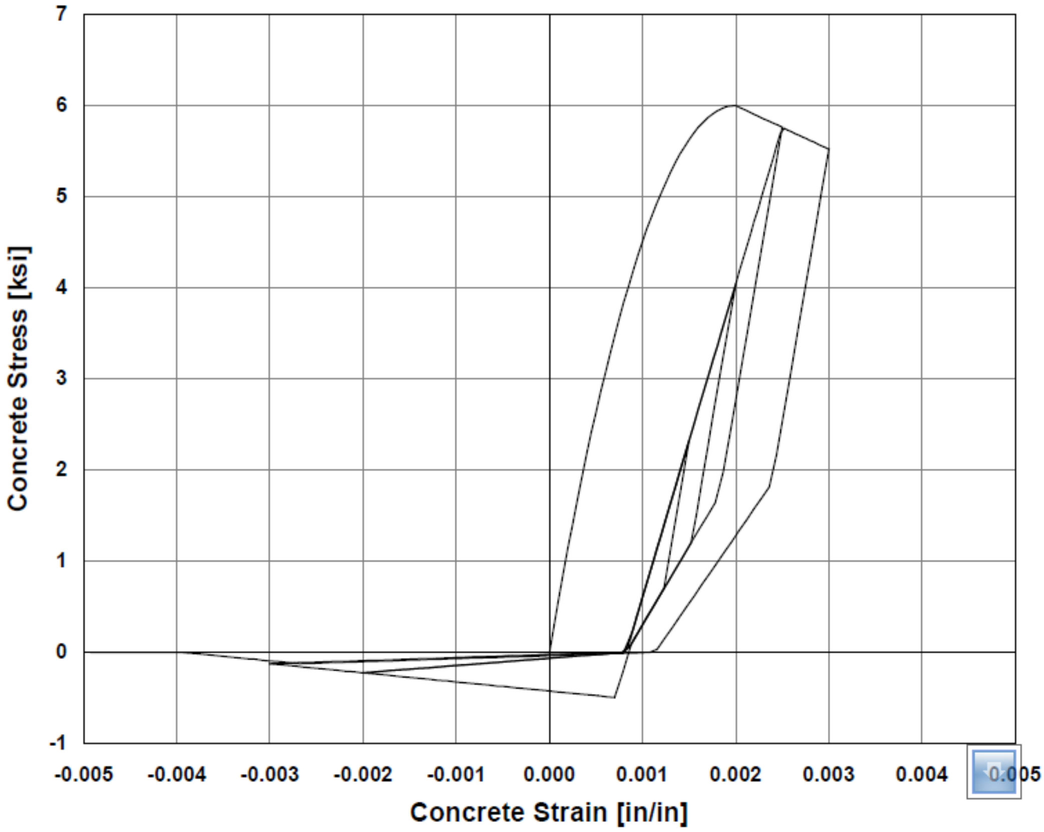


Pushover Modeling (Column Element)

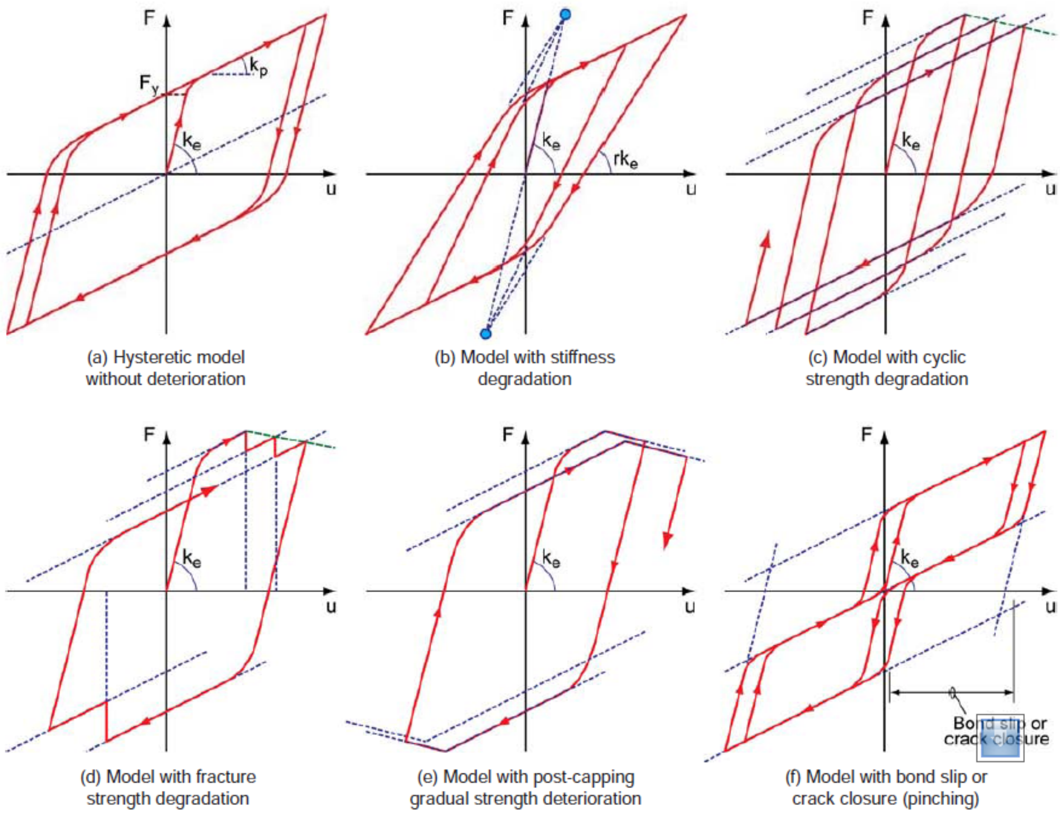
Three dimensional Column Element



Pushover Modeling (Properties)



Pushover Modeling (Properties)



Target Displacement (FEMA-356)

- Estimation of Target Displacement
  - Estimate effective elastic stiffness, K_e
  - Estimate post yield stiffness, K_s
  - Estimate effective fundamental period, T_e
  - Calculate target roof displacement

$$\delta = C_0 C_1 C_2 C_3 S_a T_e^2 /(4\pi^2) * g$$

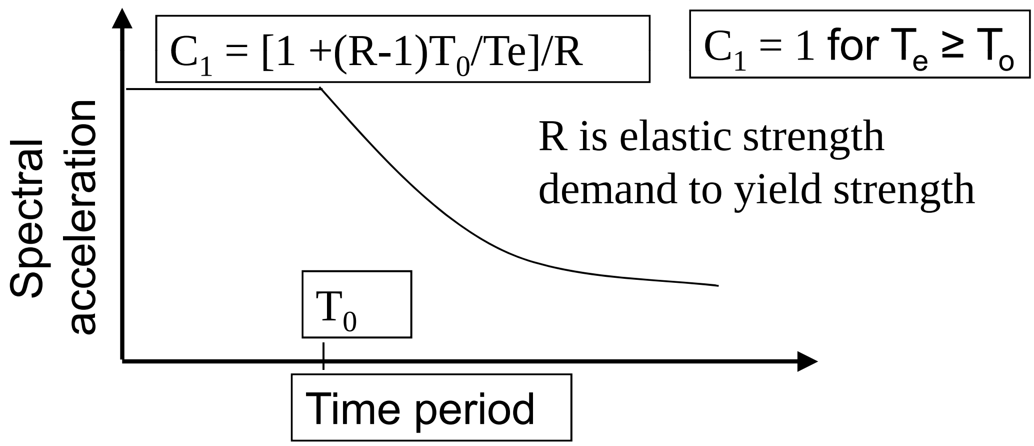
Target Displacement (FEMA-356)

- Calculation of C_0
  - Relates spectral to roof displacement
  - Use modal participation factor for control node from first mode or
  - Use modal participation factor for control node from deflected shape at the target displacement or
  - Use tables based on number of stories and varies from 1 to 1.5

Target Displacement (FEMA-356)

- Calculation of C_1

  Modifier for inelastic displacement



Target Displacement (FEMA-356)

- Calculation of C_2
  - Modifier for hysteresis loop shape
    - Depends on framing type (degrading strength)
    - Depends on performance level
    - Depends on Effective Period
    - 1.0 shall be permitted for nonlinear procedures

Target Displacement (FEMA-356)

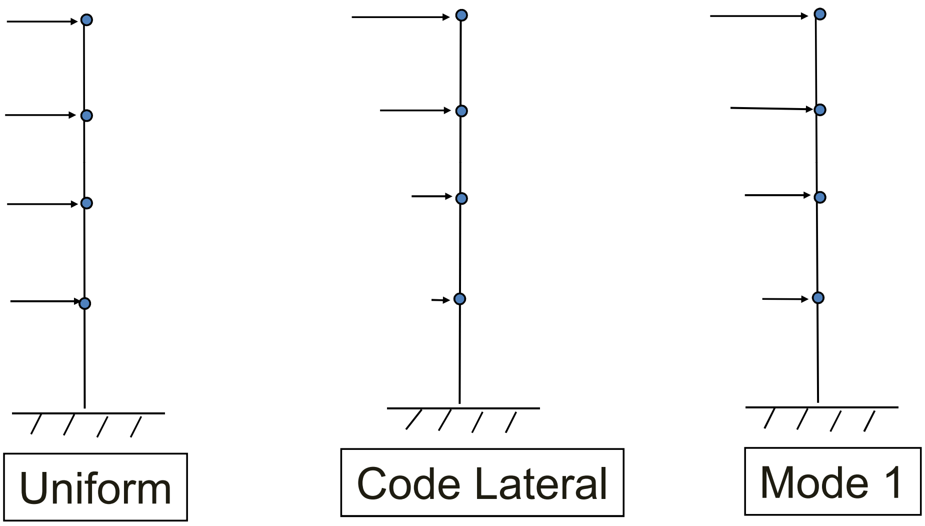
- Calculation of C_3
  - Modifier for dynamic second order effects
    - C_3 = 1 if post yield slope is positive else
    - $C_3 = 1 + [\,|\alpha|(R-1)^{3/2}\,]/T_e$
- R=Ratio of elastic strength demand to calculated yield strength
- S_a = Response spectrum acceleration

Pushover Modeling (Loads)

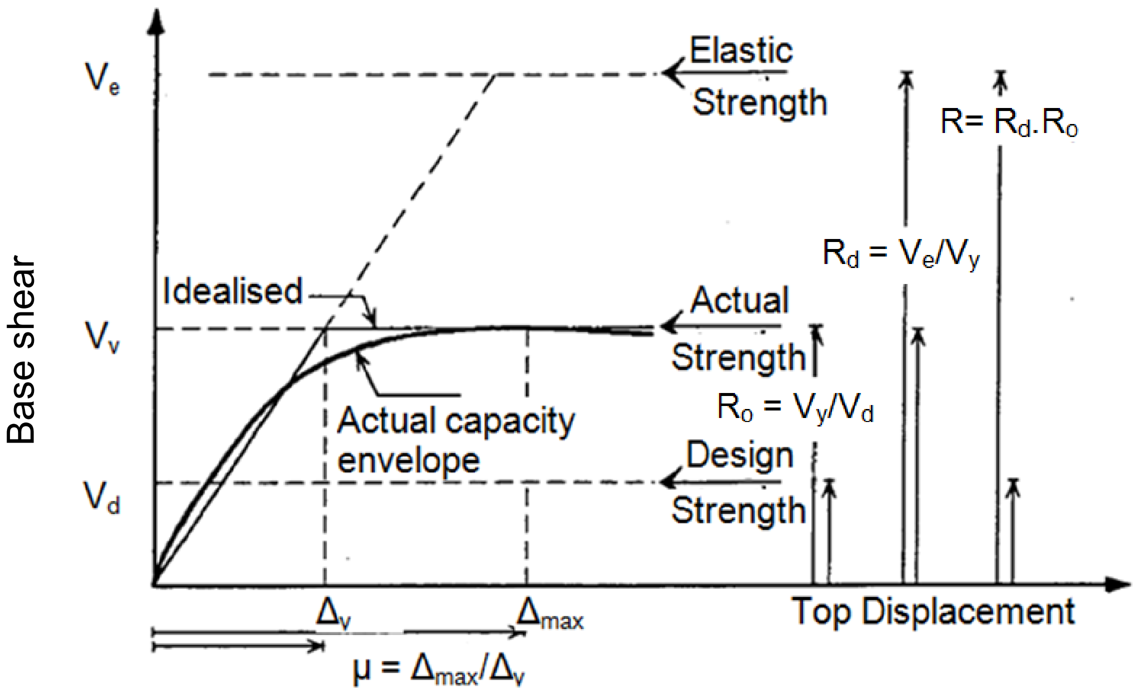
- Start with Gravity Loads
  - Dead Load
  - Some portion of Live Load
- Select Lateral Load Pattern
  - Lateral Load Patterns (Vertical Distribution)
  - Lateral Load Horizontal Distribution
  - Torsional Effects
  - Orthogonal Effects

Pushover Modeling (Loads)

Lateral Load Patterns (Vertical Distribution)

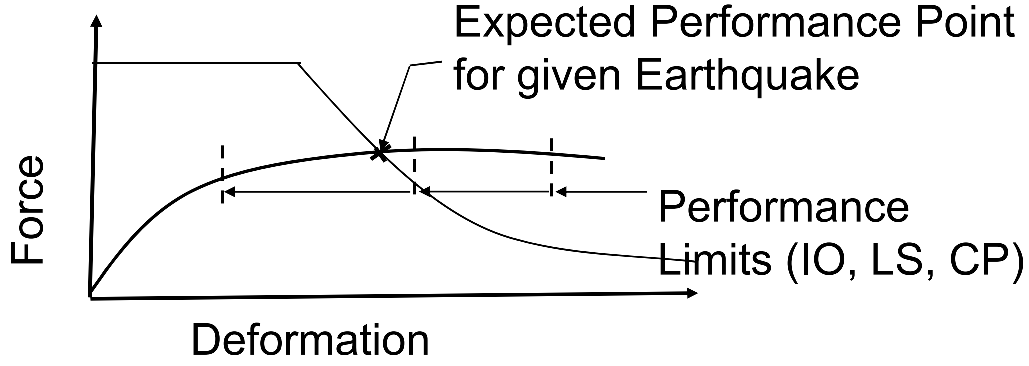


Capacity Curve



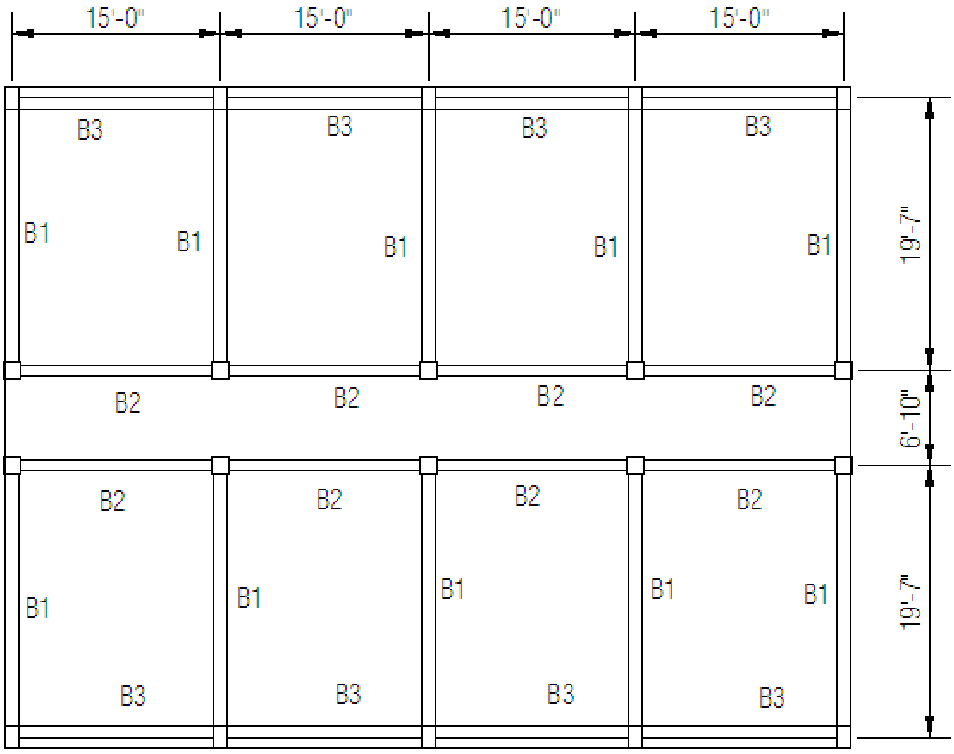
Relation of different factors

Performance Check Using Pushover



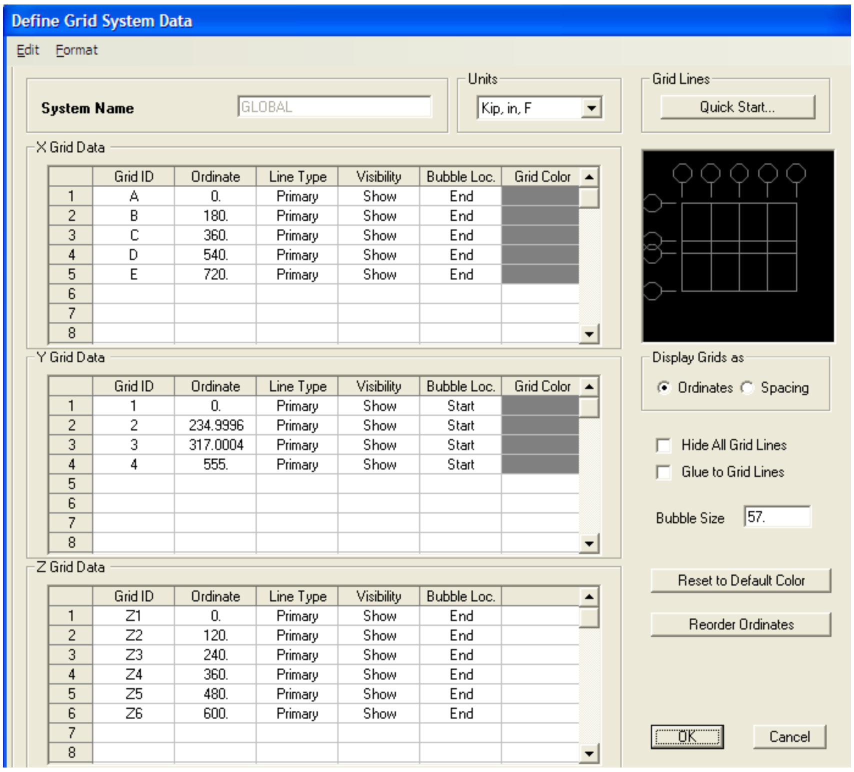
- Construct Pushover curve
- Select earthquake level(s) to check and construct their spectrum curves
- Decide the performance level(s) (i.e.: IO, LS, CP)
- Verify structural performance with guidelines
  - Capacity Spectrum Method (ATC-40)
  - Displacement Coefficient Method (FEMA 356)

Pushover Analysis Example



Clinic Building

Pushover Analysis Example



X Grid Data

	Grid ID	Ordinate	Line Type	Visibility	Bubble Loc.	Grid Color
1	A	0.	Primary	Show	End	
2	B	180.	Primary	Show	End	
3	C	360.	Primary	Show	End	
4	D	540.	Primary	Show	End	
5	E	720.	Primary	Show	End	
6						
7						
8						

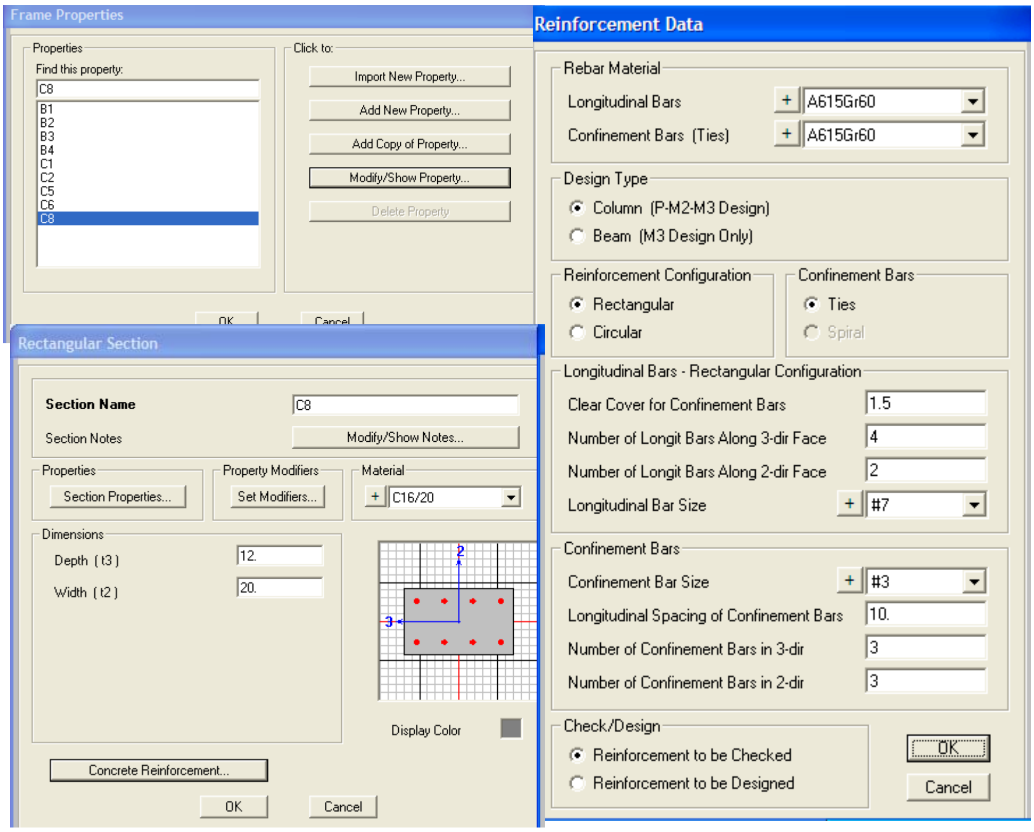
Y Grid Data

	Grid ID	Ordinate	Line Type	Visibility	Bubble Loc.	Grid Color
1	1	0.	Primary	Show	Start	
2	2	234.9996	Primary	Show	Start	
3	3	317.0004	Primary	Show	Start	
4	4	555.	Primary	Show	Start	
5						
6						
7						
8						

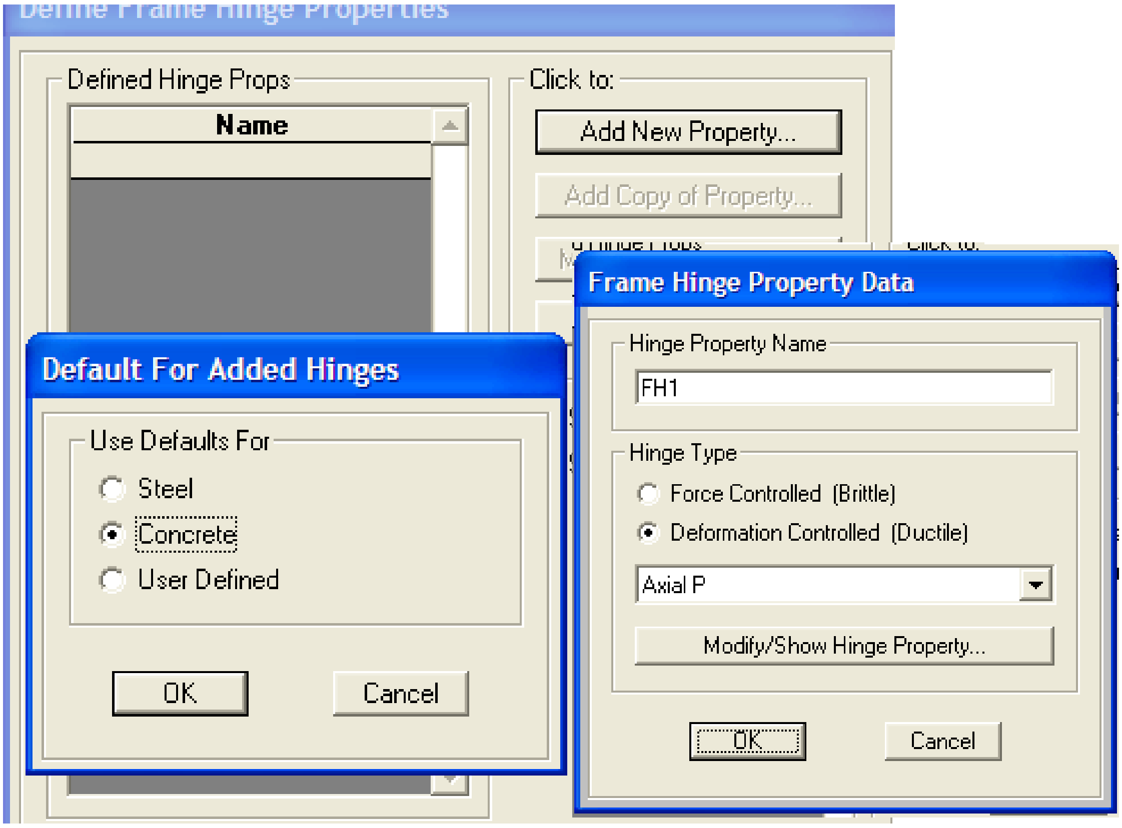
Z Grid Data

	Grid ID	Ordinate	Line Type	Visibility	Bubble Loc.
1	Z1	0.	Primary	Show	End
2	Z2	120.	Primary	Show	End
3	Z3	240.	Primary	Show	End
4	Z4	360.	Primary	Show	End
5	Z5	480.	Primary	Show	End
6	Z6	600.	Primary	Show	End
7					
8					

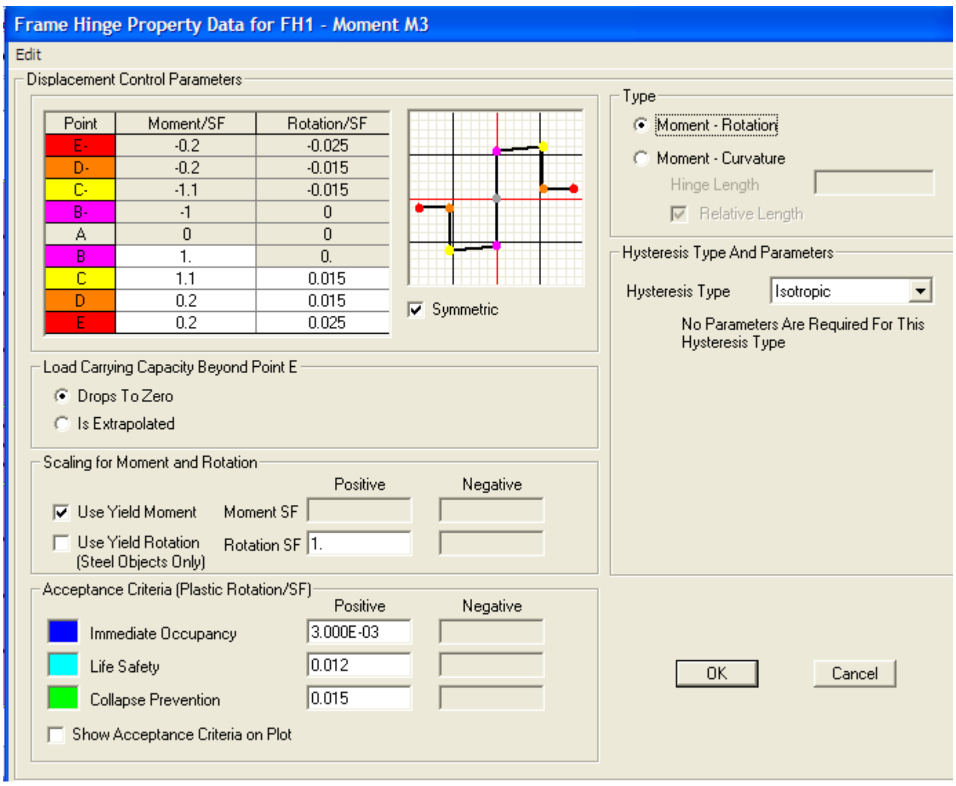
Pushover Analysis Example



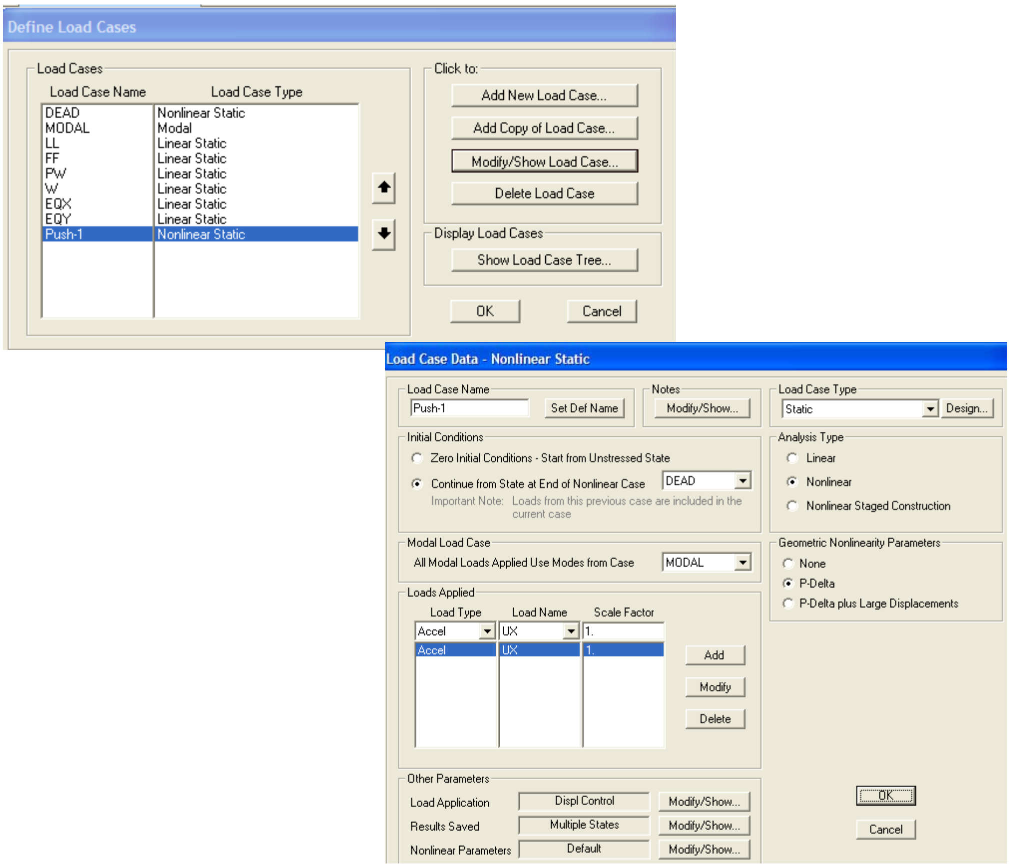
Pushover Analysis Example



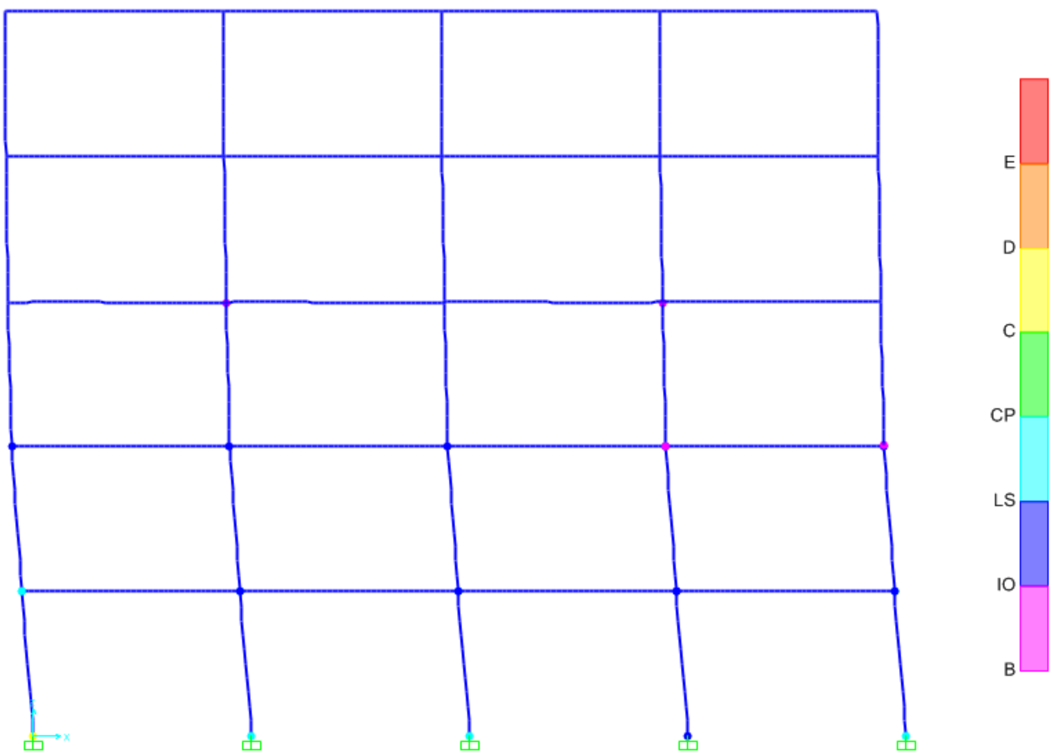
Pushover Analysis Example



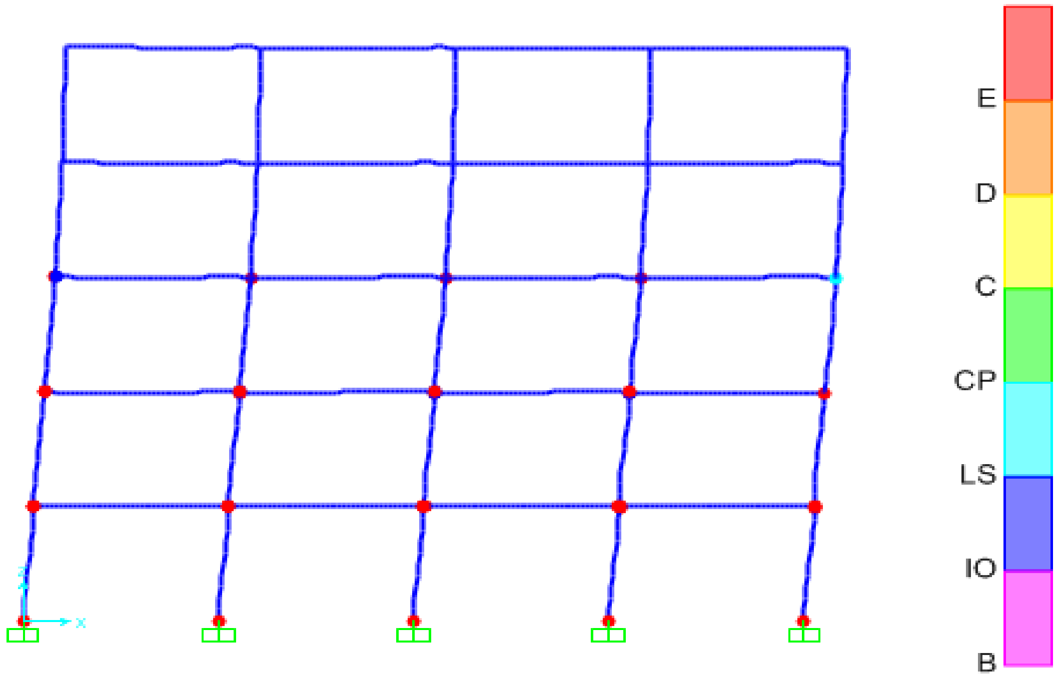
Pushover Analysis Example



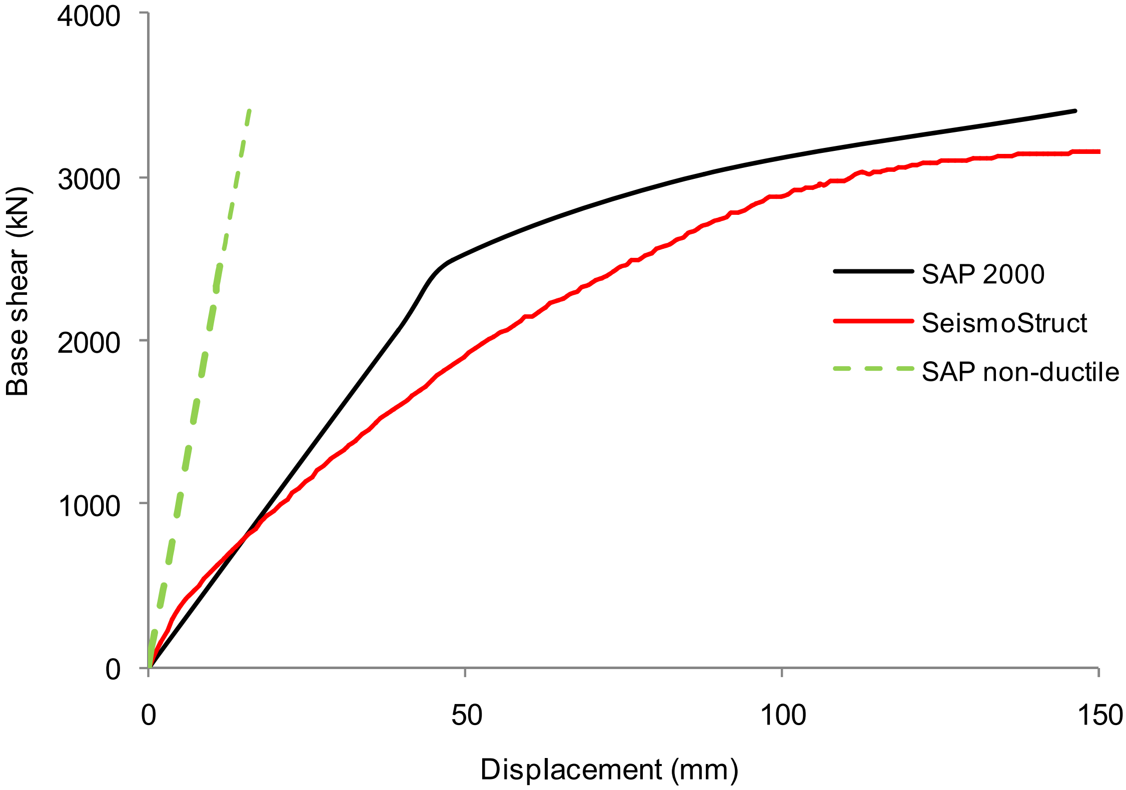
Pushover Analysis (Results)



Pushover Analysis (Results)



Pushover Analysis (Results)



SEISMIC DESIGN CONCEPT FOR NEW BUILDINGS

MD. MOMINUR RAHMAN
EXECUTIVE ENGINEER
PUBLIC WORKS DEPARTMENT AND
TEAM MEMBER, COMPONENT-2
CNCRP PROJECT.

An effective seismic design generally includes

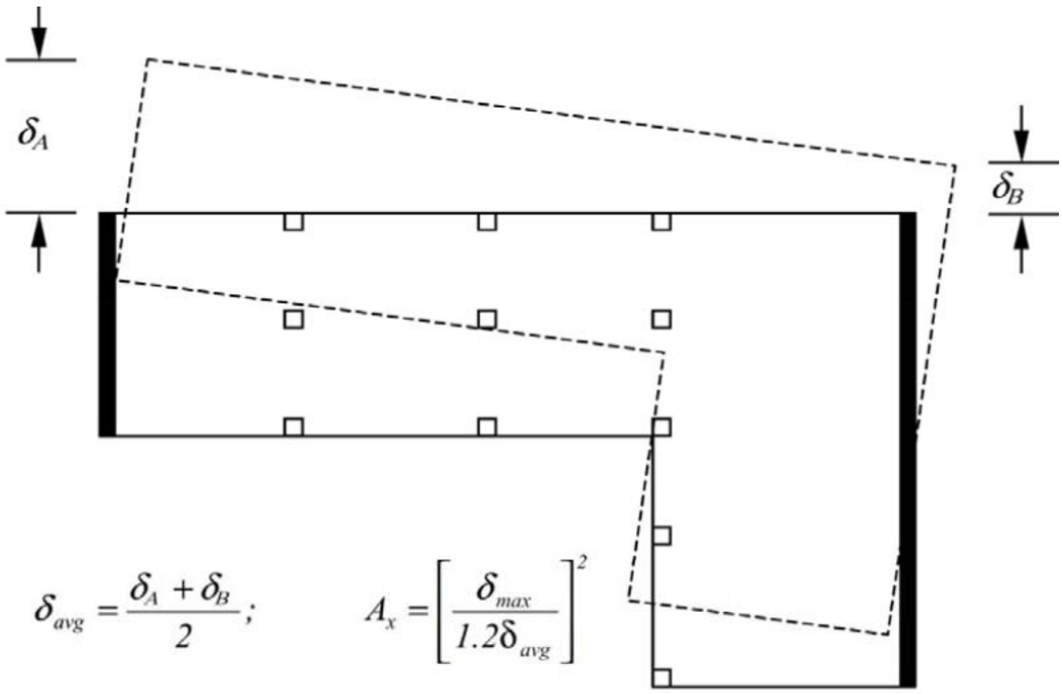
1. Layout of a lateral force-resisting system which includes providing a redundant and continuous load path to ensure that a building responds as a unit during ground motion.

2. Determination of code-prescribed forces and deformations generated by the ground motion, and distribution of the forces vertically to the lateral force-resisting system.

3. Analysis of the building for the combined effects of gravity and seismic loads to verify that adequate vertical and lateral strengths and stiffnesses are achieved to satisfy the structural performance and acceptable deformation levels prescribed in the building code.

4. Structural detailing to assure that the structure has sufficient inelastic deformability to undergo large deformations when subjected to a major earthquake.

HORIZONTAL TORSIONAL MOMENTS



The accidental torsional moment M_{tai} at level i is given as:

$M_{tai} = e_{ai} F_i$

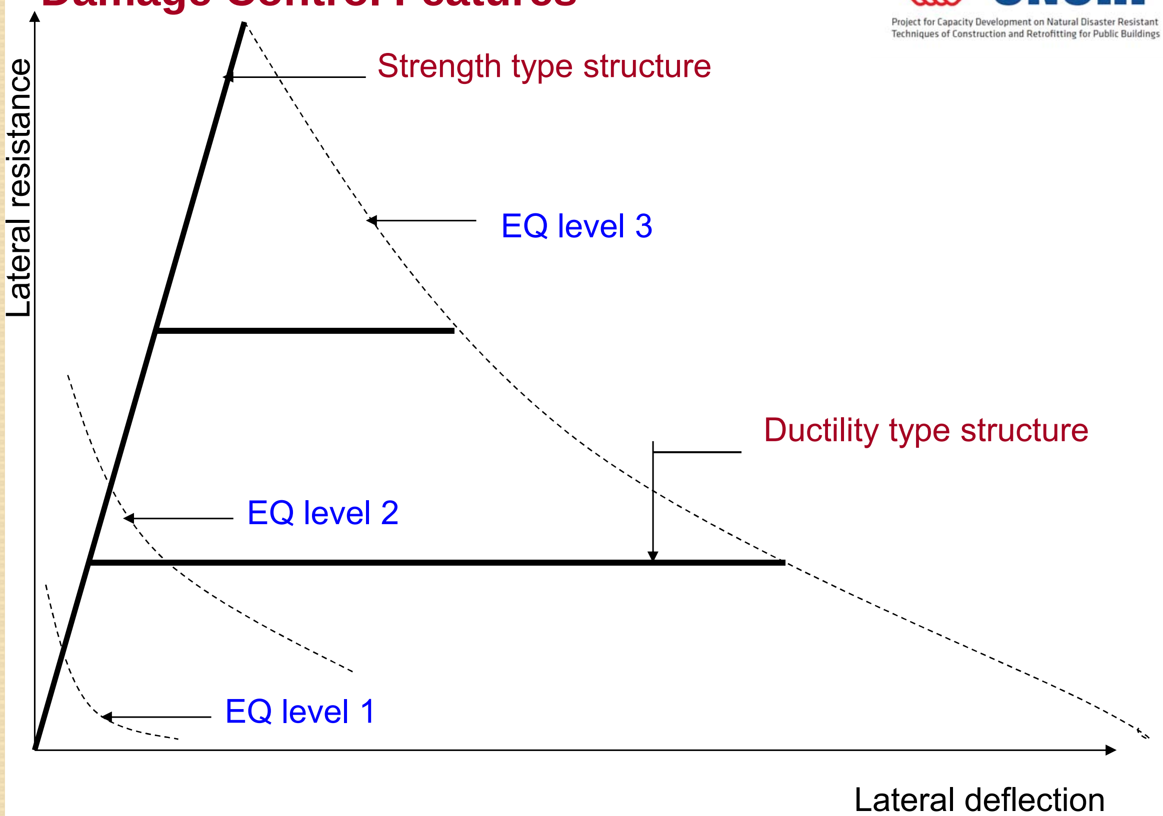
where, e_{ai} = accidental eccentricity of floor mass at level i applied in the same direction at all floors = ±0.05 Li

L_i = floor dimension perpendicular to the direction of seismic force considered.

Where torsional irregularity exists for Seismic Design Category C or D, the irregularity effects shall be accounted for by increasing the accidental torsion M_{tai} at each level by a torsional amplification factor,

$A_x = [\delta_{max}/ (1.2\delta_{avg})]^2 \leq 3.0$

Damage Control Features

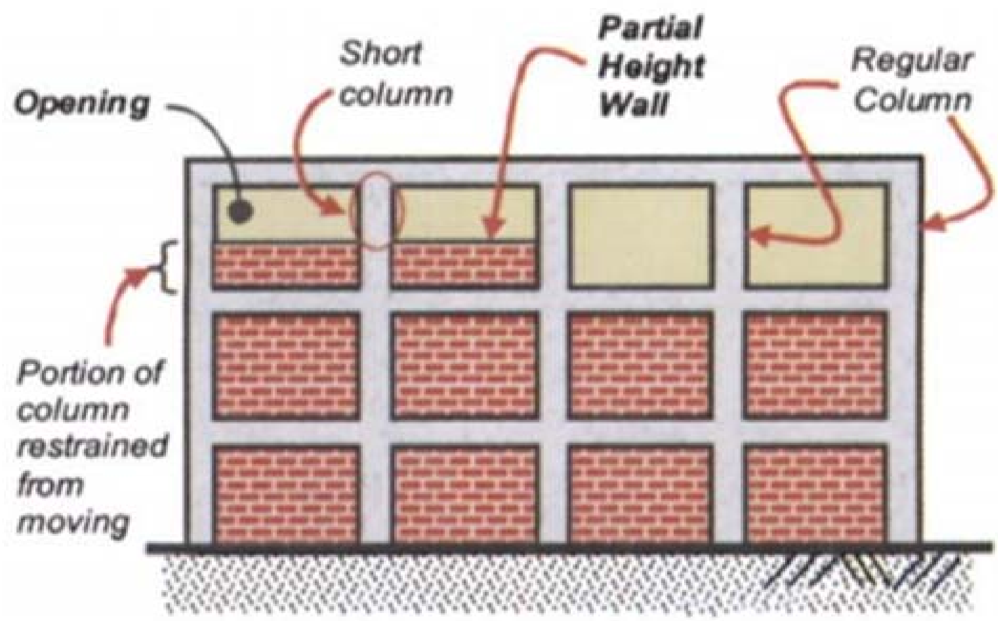


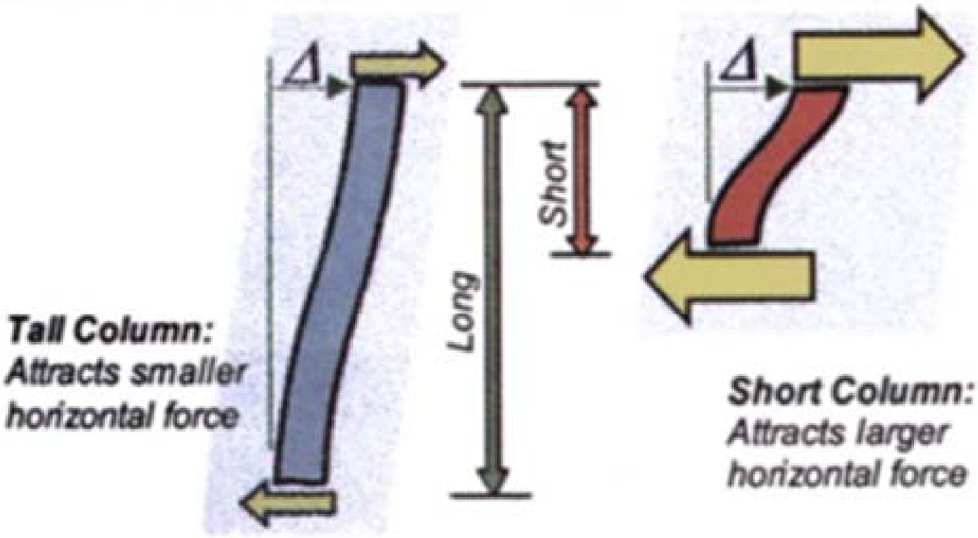
To minimize the damage of nonstructural elements, special care in detailing, either to isolate these elements or to accommodate the movement, is required.

Breakage of glass windows can be minimized by providing adequate clearance at edges to allow for frame distortions.

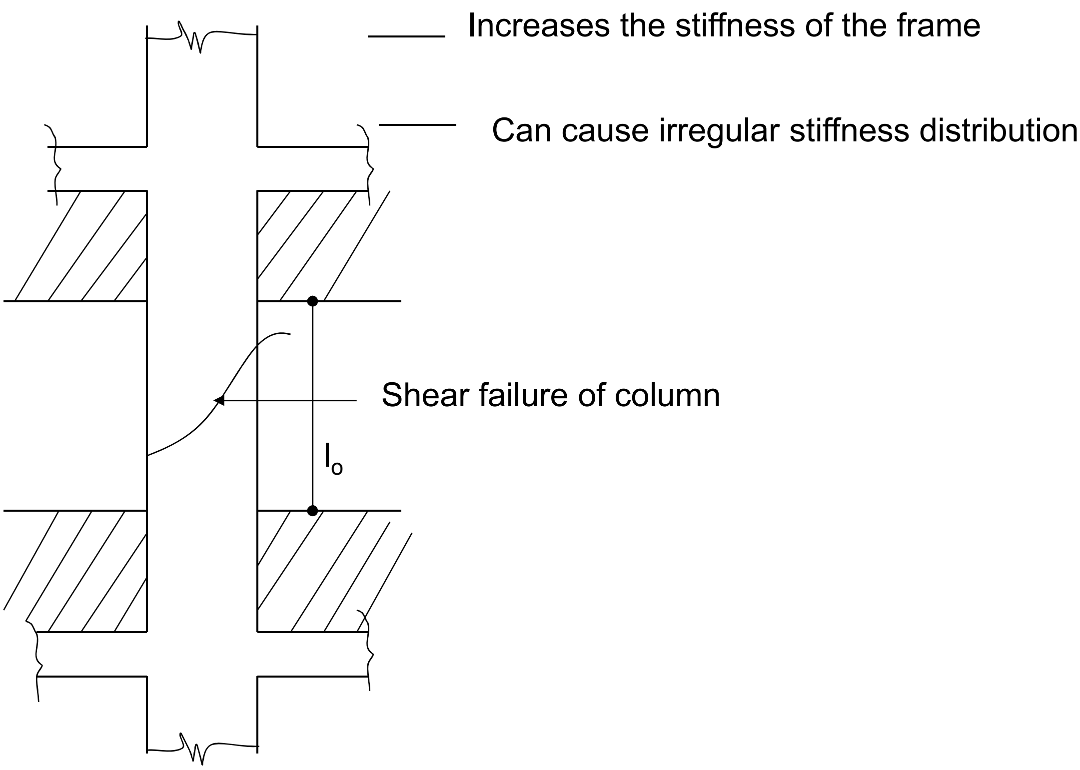
Damage to rigid nonstructural partitions can be largely eliminated by providing a detail at the top and sides, which will permit relative movement between the partitions and the adjacent structural elements.

Effect of non-structural elements





Poor behaviour of short columns is due to the fact that in an earthquake, a tall column and a short column of same cross section move horizontally by same amount which can be seen from the given figure.



Progressive collapse of a brittle structure

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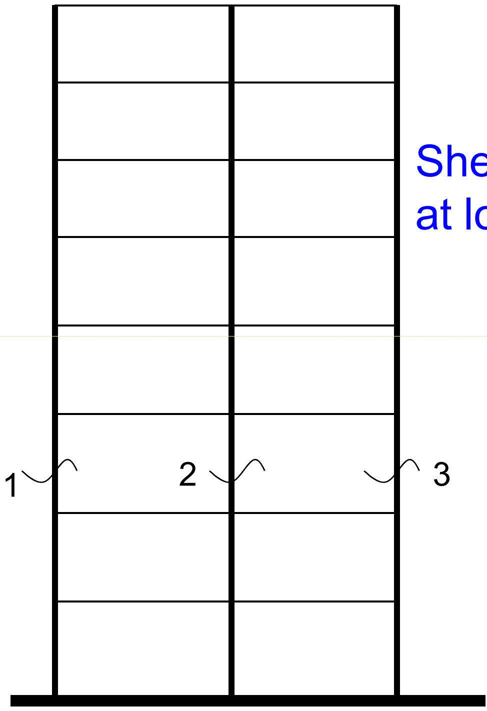

Shear failure of short column occurs at low deformation.

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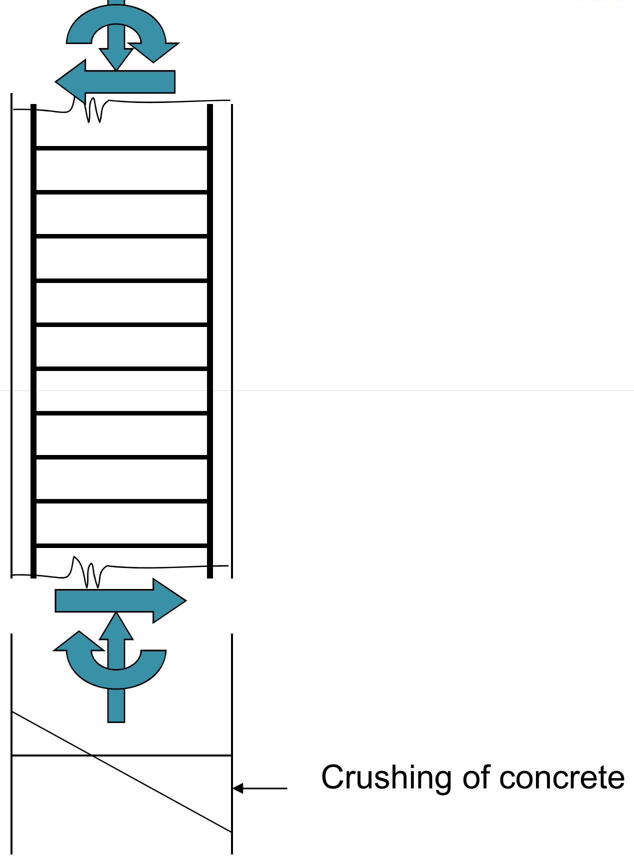
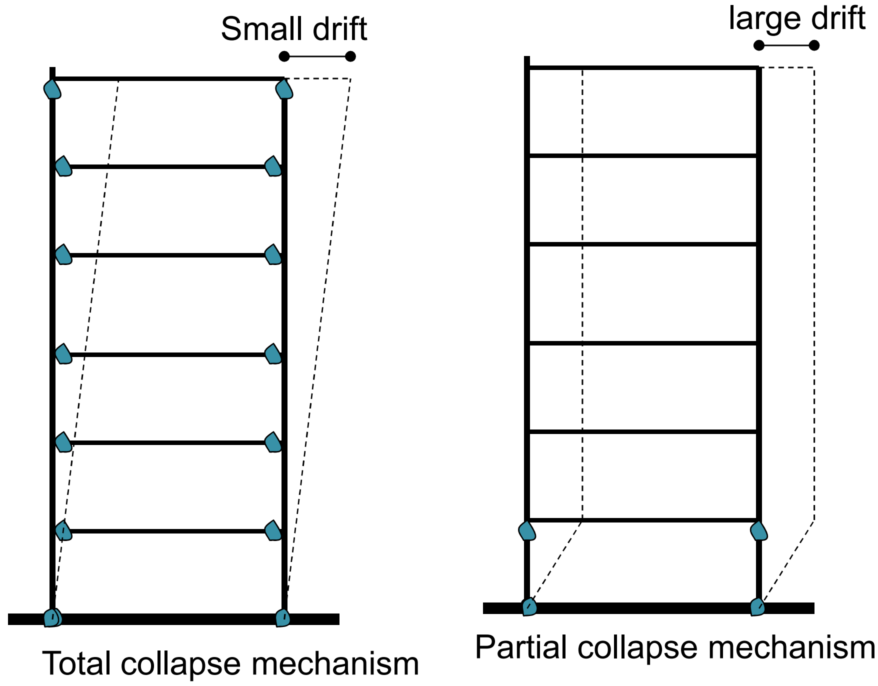
Short column failure

Flexural compression failure of corner colum





Soft story failure

Effect of column tie on shear failure of column

1. If column tie is absent, brittle shear failure occurs in diagonal tension mode.

2. If minimum column tie is present, diagonal compression failure of concrete occurs after tie yielding. Not brittle failure, but deformation capacity is low.

3. Tie resists tension under shear and must be 135^0 hook.

•Pounding effect

•Deterioration with age

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Beam-column joint failure

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Splicing failure of column main reinforcement

Poor quality of concrete

SITE INVESTIGATION

Appropriate site investigations should be carried out to identify the ground conditions influencing the seismic action. The ground conditions at the building site should normally be free from risks of ground rupture, slope instability and permanent settlements caused by liquefaction or densification during an earthquake. The possibility of such phenomena should be investigated in accordance with standard procedures.

Liquefaction potential and possible consequences should be evaluated for design earthquake ground motions consistent with peak ground accelerations. Any settlement due to densification of loose granular soils under design earthquake motion should be studied. The occurrence and consequences of geologic hazards such as slope instability or surface faulting should also be considered. The dynamic lateral earth pressure on basement walls and retaining walls during earthquake ground shaking is to be considered as an earthquake load for use in design load combinations.

Foundation failure due to liquefaction

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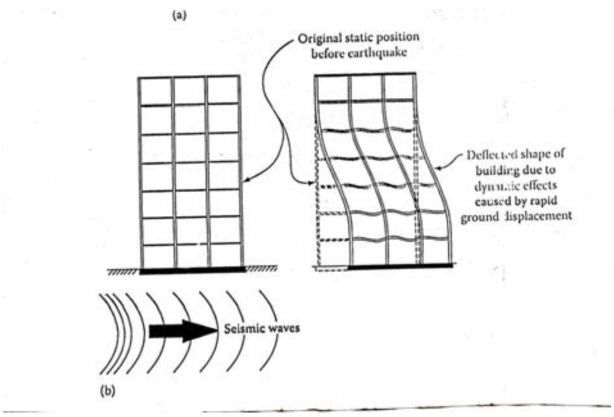
Structural Response

The inertia forces generated by the horizontal components of ground motion require greater consideration for seismic design since adequate resistance to vertical seismic loads is usually provided by the member capacities required for gravity load design.



Load Path

1. There must be a complete gravity and lateral force-resisting system that forms a continuous load path between the foundation and all portions of the building.

2. If there is a discontinuity in the load path, the building is unable to resist seismic forces regardless of the strength of the elements.

3. Interconnecting the elements needed to complete the load path is necessary to achieve the required seismic performance.

Ductility

Ductility is the capacity of building materials, systems or structures to absorb energy by deforming into the inelastic range. The capability of a structure to absorb energy, with acceptable deformations and without failure, is a very desirable characteristic in any earthquake-resistant design.

Ductility or hysteretic behavior may be considered as an energy-dissipating mechanism due to inelastic behavior of the structure at large deformations.

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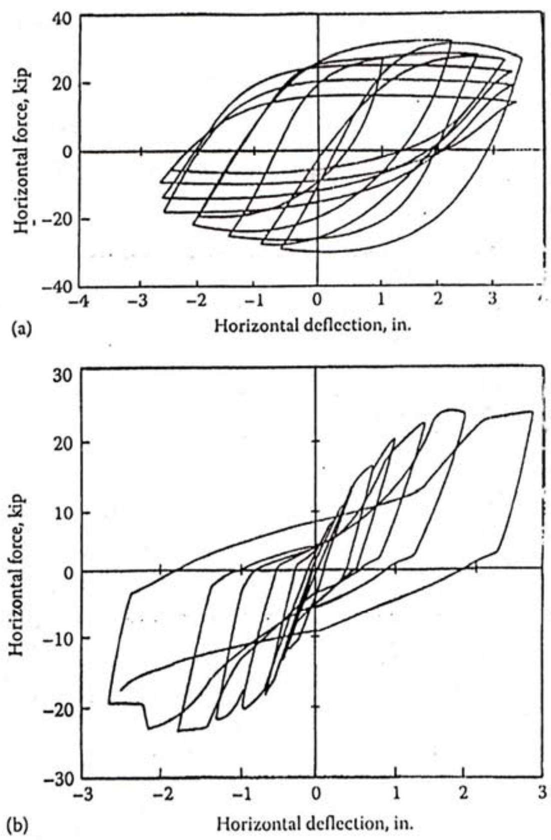

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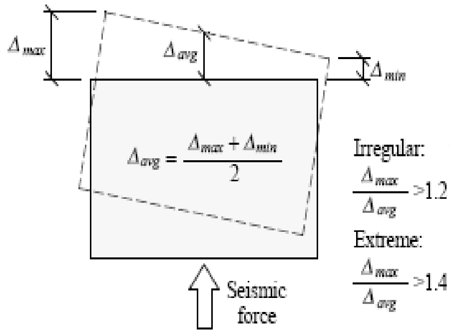
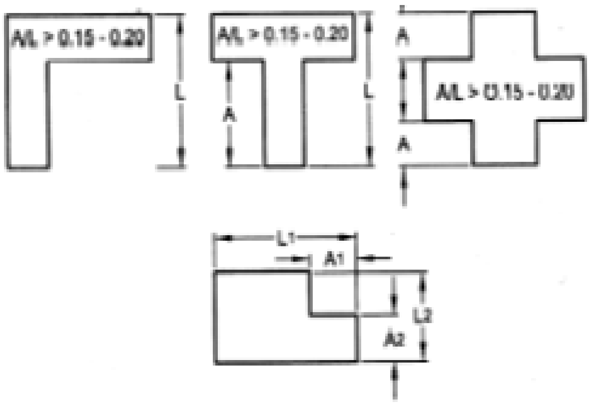
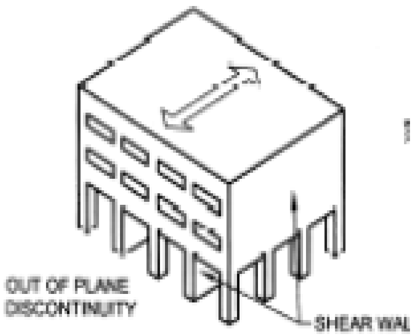
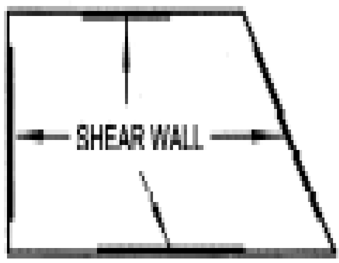
Irregular Buildings

Geometric configuration, type of structural members, details of connections, and materials of construction, all have a profound effect on the structural dynamic response of a building. When a building has irregular features, such as asymmetry in plan or vertical discontinuity, the assumptions used in developing seismic criteria for buildings with regular features may not apply. So it is best to avoid creating buildings with irregular features.









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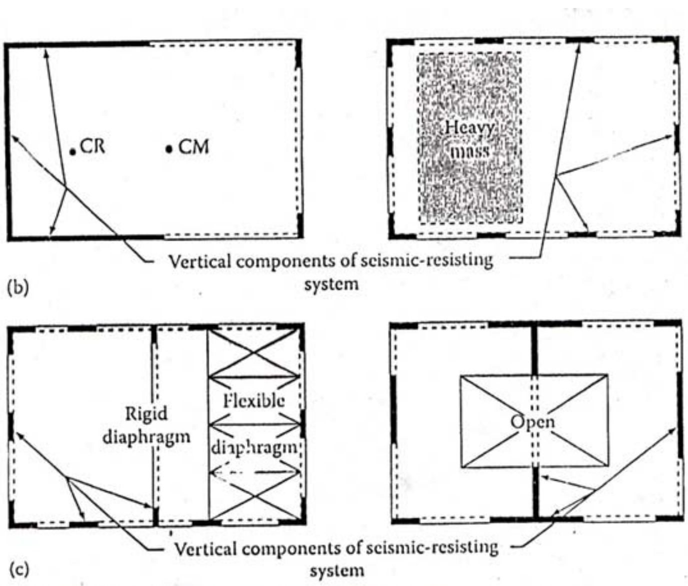

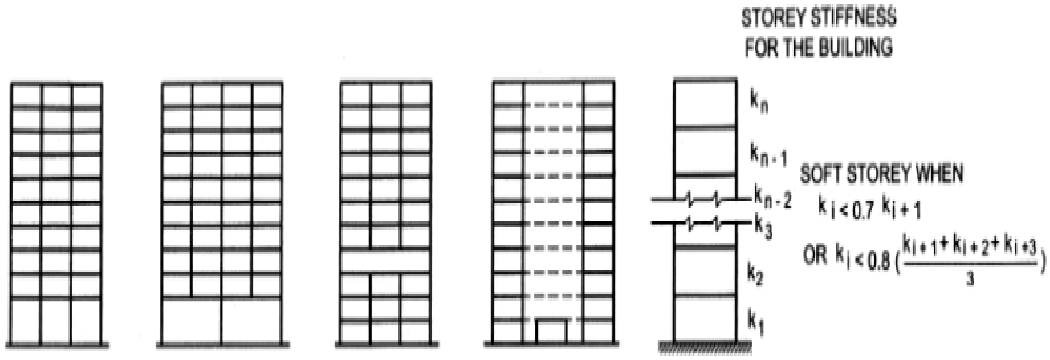

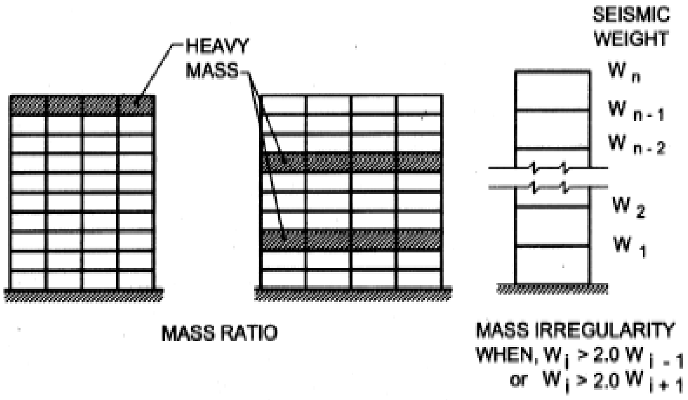

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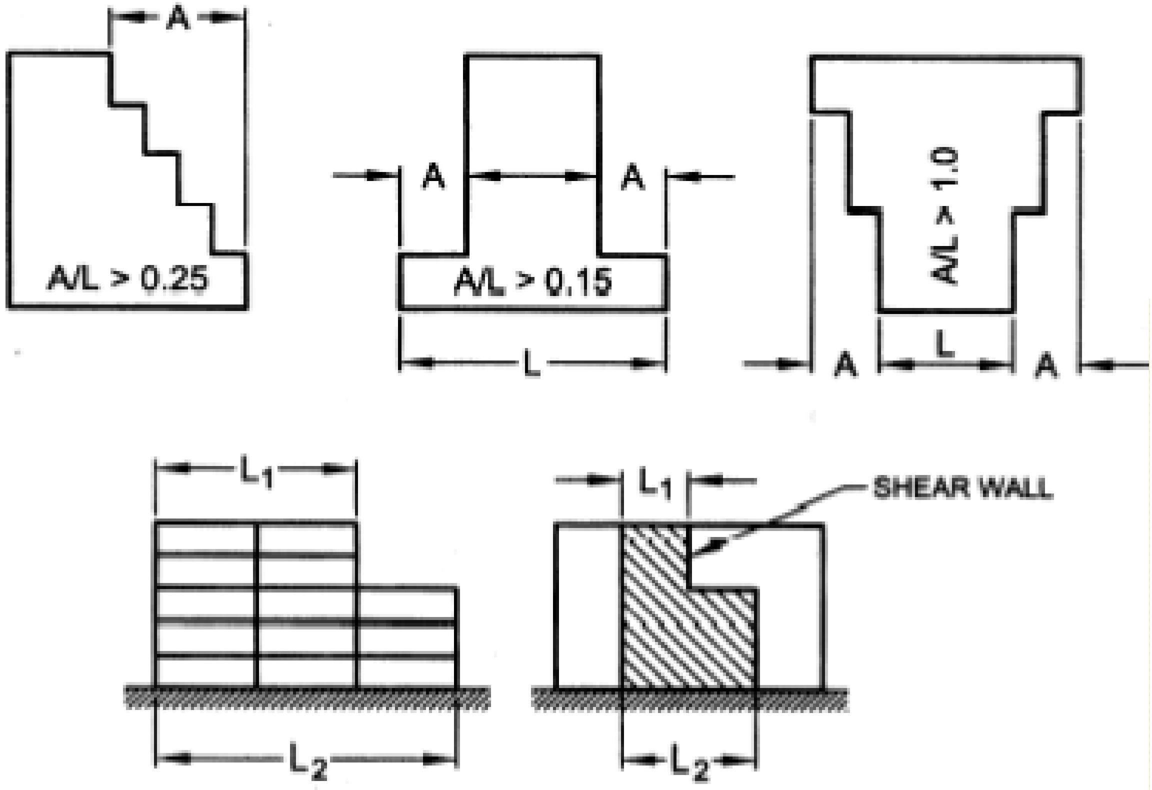
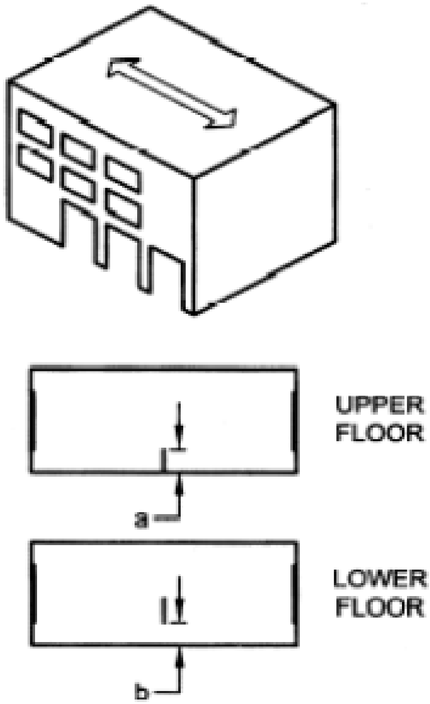
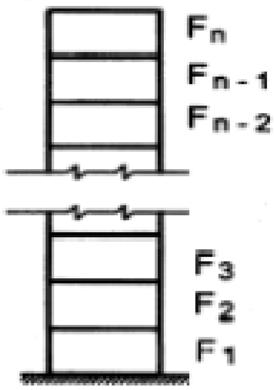




IN - PLANE DISCONTINUITY VERTICAL ELEMENTS RESISTING LATERAL FORCE, WHEN b > a

STOREY STRENGTH (LATERAL)



WEAK STOREY, WHEN $F_i < 0.8 F_{i+1}$

Redundancy

A high degree of redundancy accompanied by redistribution capacity through ductility is desirable, enabling a more widely spread energy dissipation across the entire structure and an increased total dissipated energy. The use of evenly distributed structural elements increases redundancy.

The failure of a single connection or component in a building with a redundant system does not adversely affect its lateral stability.

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Lateral Force-Resisting Systems

In moment frames, the drift may be large. So a moment-frame building can have substantial nonstructural damage and still be structurally safe.

A shear-wall building is typically more rigid than a framed structure.

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Table 2.5.7 Response reduction factor, deflection amplification factor for different Structural Systems and height limitations (m) for different seismic design categories

Seismic Force–Resisting System	Response Reduction Factor, R	Deflection Amplification Factor, C_d	Seis. Design Category B	Seis. Design Category C	Seis. Design Category D
			Height limit (m)		
A. BEARING WALL SYSTEMS (no frame)					
1. Special reinforced concrete shear walls	5	5	NL	NL	50
2. Ordinary reinforced concrete shear walls	4	4	NL	NL	NP
3. Ordinary reinforced masonry shear walls	2	1.75	NL	50	NP
4. Ordinary plain masonry shear walls	1.5	1.25	18	NP	NP
B. BUILDING FRAME SYSTEMS (with bracing or shear wall)					
1. Steel eccentrically braced frames, moment resisting connections at columns away from links	8	4	NL	NL	50
2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	7	4	NL	NL	50
3. Special steel concentrically braced frames	6	5	NL	NL	50
4. Ordinary steel concentrically braced frames	3.25	3.25	NL	NL	11
5. Special reinforced concrete shear walls	6	5	NL	50	50
6. Ordinary reinforced concrete shear walls	5	4.25	NL	NL	NP
7. Ordinary reinforced masonry shear walls	2	2	NL	50	NP
8. Ordinary plain masonry shear walls	1.5	1.25	18	NP	NP

Table 2.5.7 Response reduction factor, deflection amplification factor for different Structural Systems and height limitations (m) for different seismic design categories

Seismic Force–Resisting System	Response Reduction Factor, R	Deflection Amplification Factor, C_d	Seis. Design Category B	Seis. Design Category C	Seis. Design Category D
			Height limit (m)		
C. MOMENT RESISTING FRAME SYSTEMS (no shear wall)					
1. Special steel moment frames	8	5.5	NL	NL	P
2. Intermediate steel moment frames	4.5	4	NL	NL	35
3. Ordinary steel moment frames	3.5	3	NL	NL	NP
4. Special reinforced concrete moment frames	8	5.5	NL	NL	NL
5. Intermediate reinforced concrete moment frames	5	4.5	NL	NL	NP
6. Ordinary reinforced concrete moment frames	3	2.5	NL	NP	NP
D. DUAL SYSTEMS: SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)					
1. Steel eccentrically braced frames	8	4	NL	NL	NL
2. Special steel concentrically braced frames	7	5.5	NL	NL	NL

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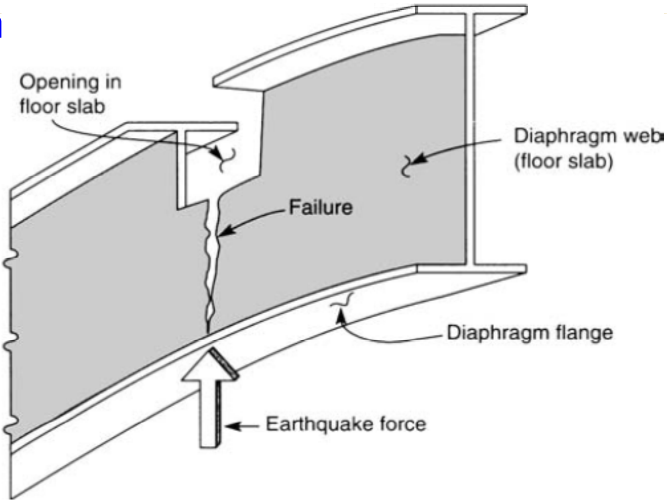
Seismic Force–Resisting System	Response Reduction Factor, R	Deflection Amplification Factor, C_d	Seis. Design Category B	Seis. Design Category C	Seis. Design Category D
			Height limit (m)		
3. Special reinforced concrete shear walls	7	5.5	NL	NL	NL
4. Ordinary reinforced concrete shear walls	6	5	NL	NL	NP
E. DUAL SYSTEMS: INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)					
1. Special steel concentrically braced frames	6	5	NL	NL	11
2. Special reinforced concrete shear walls	6.5	5	NL	NL	50
3. Ordinary reinforced masonry shear walls	3	3	NL	50	NP
4. Ordinary reinforced concrete shear walls	5.5	4.5	NL	NL	NP
F. DUAL SHEAR WALL-FRAME SYSTEM: ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	4.5	4	NL	NP	NP
G. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE	3	3	NL	NL	NP

Diaphragms

Earthquake loads at any level of a building will be distributed to the lateral load-resisting vertical elements through the floor slabs. Inappropriate location or large size openings for stairs or lift cores create problems similar to those related to cutting the flanges and holes in the web of a steel beam adjacen



Proposed BNBC 2010:
Seismic Design Category: Structural Implications

Seismic design category D has the most stringent seismic design detailing ,while seismic design category B has the least seismic design detailing requirements. Certain structural systems are not permitted for seismic design categories C and D.

	Imortance Class I and II				Imortance Class III and IV			
Site class	Zone 1	Zone 2	Zone 3	Zone 4	Zone 1	Zone 2	Zone 3	Zone 4
SA	B	C	C	D	C	D	D	D
SB	B	C	D	D	C	D	D	D
SC	B	C	D	D	C	D	D	D
SD	C	D	D	D	D	D	D	D
SE, S1, S2	D	D	D	D	D	D	D	D

Dynamic Analysis

For the buildings that are asymmetrical or with areas of discontinuity or irregularity, dynamic analysis is used to determine significant response characteristics such as (a) the effects of the structure's dynamic characteristics on the vertical distribution of lateral forces, (b)the increase in dynamic loads due to torsional motions, (c) the influence of higher modes, resulting in an increase in story shears and deformations.

Static methods specified in building codes are based on single-mode response with simple corrections for including higher mode effects.While appropriate for simple regular structures, the simplified procedures do not take into account the full range of seismic behavior of complex structures. Therefore dynamic analysis is the preferred method for the design of buildings with unusual or irregular geometry.

REQUIREMENT FOR DYNAMIC ANALYSIS

Dynamic analysis should be performed to obtain the design seismic force, and its distribution to different levels along the height of the building and to the various lateral load resisting elements, for the following buildings:

a) Regular buildings with height greater than 40 m in Zones 2, 3, 4 and greater than 90 m in Zone 1.

b) Irregular buildings with height greater than 12 m in Zones 2,3, 4 and greater than 40 m in Zone 1. For irregular buildings, smaller than 40 m in height in Zone 1, dynamic analysis, even though not mandatory, is recommended.

P-DELTA EFFECTS:

The P - delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered if the stability coefficient (θ) determined by the following equation is not more than 0.10:

$\theta = P_x \Delta /(V_x h_{sx} C_d)$

Where,

Px = the total vertical design load at and above level x; where computing *Px*, no individual load factor need exceed 1.0

Δ = the design story drift occurring simultaneously with *Vx*

Vx = the storey shear force acting between levels *x* and *x* − 1

hsx = the story height below level *x*

Cd = the deflection amplification factor given in BNBC

The stability coefficient (θ) shall not exceed θmax.

THANK YOU

Code Provisions for Seismic Analysis and Design - Example

[Short Training Course on Seismic Assessment, Retrofit Design and Construction of RC Buildings
11-20 Feb, 2013]

Md. Rafiqul Islam
Executive Engineer
PWD Design Division – 3
&
Team Leader
Working Team – 2
CNCRP Project

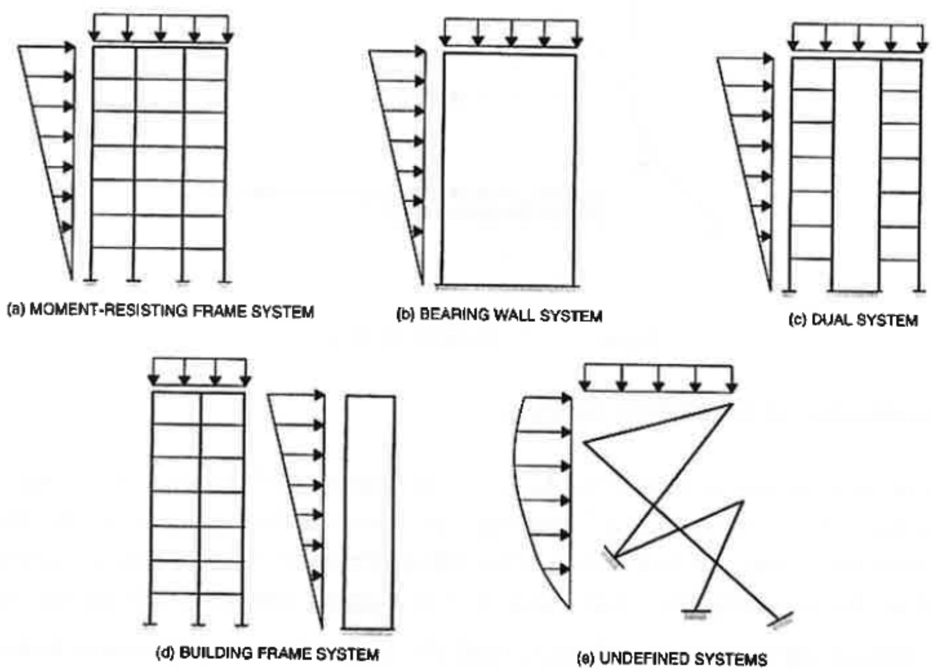
Earthquake Design Philosophy

1. It is uneconomical and unnecessary to design a structure in elastic range for maximum EQ induced inertia force.
2. The large deformation during EQ will be accompanied by yielding in some of the members of the structure.
3. Critical regions of certain members should have sufficient inelastic deformability to dissipate seismic energy.
4. Structure will not collapse when subjected to several cycles of loading into inelastic range.
5. Proper rebar detailing should avoid all forms of brittle failure.

Considerations for EQ Analysis

1. Selection of lateral force resisting system.
2. Check irregularities of structure.
3. Occupancy type of structure.
4. Location of structure in seismic zoning map.
5. Subsoil characteristics

Seismic Force Resisting Structural System



Reference: 'Seismic and Wind Design of Concrete Buildings – S. K. Gosh and Qiang Shen

Types of Moment Frame

Moment Frame: A frame in which member and joint resist lateral forces by flexure.

- Ordinary Moment Frame
- Intermediate Moment Frame
- Special Moment Frame

Ductility is the capacity of building material, systems or structure to absorb energy by deforming into inelastic range.

Choice of Frame (or SDC)

- Restriction from Code
  - Location of building
  - Occupancy type
  - Height of building
  - Soil type
- Choice of the client or designer

✓ Designer must confirm all the provisions of Code of specific frame type.

✓ Site engineer must ensure design and detailing provided by the designer.

Calculation of EQ force

Design base shear $V = S_a W$

S_a = Lateral seismic force coefficient
W = Total seismic weight of the building

In addition to total dead load, consideration for live load are:

a) Live load ≤ 3.0 KN/m^2, consider minimum 25% of live load
b) Live load ≥ 3.0 KN/m^2, consider minimum 50% of live load
c) 100% of permanent heavy equipment or retained liquid or any imposed load

Building Codes Implied Performance

	Return Period
• Ability to resist frequent, minor earthquakes without damage	100 yrs
• Ability to resist infrequent, moderate earthquakes with limited structural and nonstructural damage	475 yrs
• Ability to resist worst earthquakes ever likely to occur without collapse or major life safety endangerment	2475 yrs

Basic consideration:
Design Basis Earthquake (DBE) ground motion = 2/3 of Maximum Considered Earthquake (MCE) ground motion

Design Spectral Acceleration

$$S_a = \frac{2}{3}\frac{ZI}{R}C_s \leq \frac{2}{3}ZI\beta$$

Z = Seismic zone coefficient

I = Structure importance actor

R = Response reduction factor

β = Coefficient for lower bound of S_a = 0.2

C_s = Normalized acceleration response spectrum (function of structure period and soil type)

$$\frac{I}{R} \leq 1.0$$

Site Classification

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity $\bar{V}_s$ (m/s)	Standard Penetration Value, $\bar{N}$ (blows/30cm)	Undrained shear strength, $\bar{S}_u$ (kPa)
SA	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	--	--
SB	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
SC	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
SD	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70

Site Classification

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity $\bar{V}_s$ (m/s)	Standard Penetration Value, $\bar{N}$ (blows/30cm)	Undrained shear strength, $\bar{S}_u$ (kPa)
SE	A soil profile consisting of a surface alluvium layer with V_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $V_s > 800$ m/s.	--	--	--
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	--	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types SA to SE or S_1	--	--	--

Normalized Acceleration Response Spectrum (C_s)

$$C_s = S\left(1 + \frac{T}{T_B}(2.5\eta - 1)\right) \quad \text{for} \quad 0 \le T \le T_B$$

$$C_s = 2.5S\eta \quad \text{for} \quad T_B \le T \le T_C$$

$$C_s = 2.5S\eta\left(\frac{T_C}{T}\right) \quad \text{for} \quad T_C \le T \le T_D$$

$$C_s = 2.5S\eta\left(\frac{T_C T_D}{T^2}\right) \quad \text{for} \quad T_D \le T \le 4\,\text{sec}$$

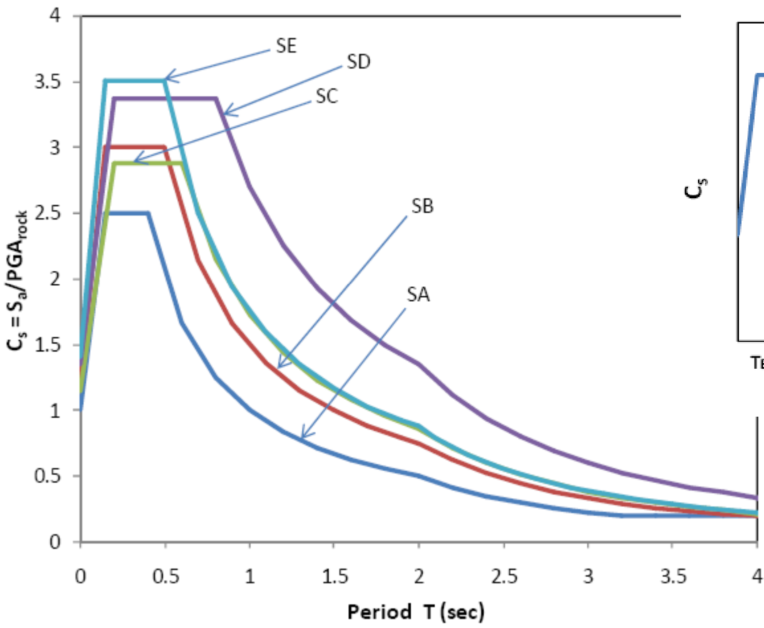
Soil type	*S*	T_B(s)	T_C (s)	T_D (s)
SA	1.0	0.15	0.40	2.0
SB	1.2	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.4	0.15	0.50	2.0

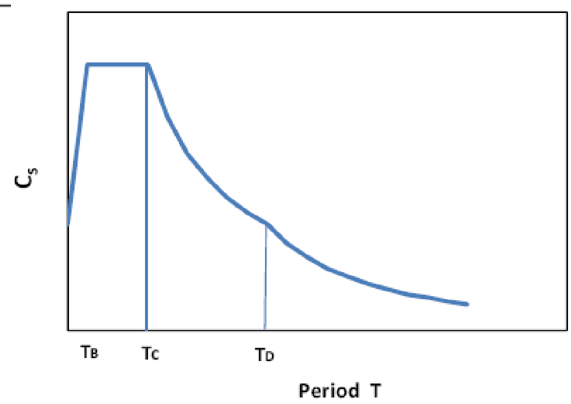
S (soil factor), T_B, T_C, T_D depends on site class

Damping correction factor, $\eta = \sqrt{10/(5+\xi)} \ge 0.55$

ξ = Damping ratio

Normalized Acceleration Response Spectrum Graph





Seismic Design Category (SDC)

Building have to be assigned a SDC based on:

- Seismic zone
- Local site condition
- Importance class

SDC – D has the most intrinsic seismic design detailing and SDC –B has the least seismic detailing requirement

	Occupancy Category I, II and III				Occupancy Category IV			
Site Class	Zone 1	Zone 2	Zone 3	Zone 4	Zone 1	Zone 2	Zone 3	Zone 4
SA	B	C	C	D	C	D	D	D
SB	B	C	D	D	C	D	D	D
SC	B	C	D	D	C	D	D	D
SD	C	D	D	D	D	D	D	D
SE, S_1, S_2	D	D	D	D	D	D	D	D

Occupancy Importance Factor (I)

Nature of Occupancy	Occupancy Category	Importance Factor
Building have low hazard to human life in the event of failure	I	1.0
Buildings except those listed in Occupancy Categories in I, III and IV	II	1.0
• Building have substantial hazard to human life in the event of failure • Buildings potential to cause a substantial economic impact or mass disruption to day to day civilian life in the event of failure • Building containing substantial quantities of toxic or explosive substances	III	1.25
Building designated as essential facilities: • Hospital, emergency shelter, power generation station • Fire, police station and emergency vehicle garage • Aviation control tower etc.	IV	1.5

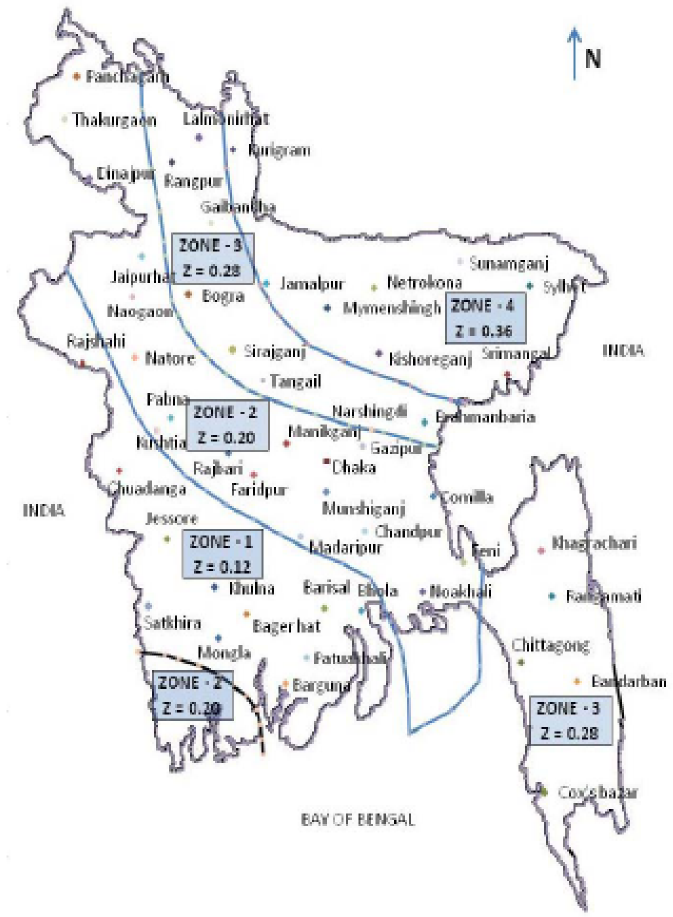
Choice of Structural System

Seismic Force Resisting System	R	C_d	SDC-B	SDC-C	SDC-D
			Height Limit (m)		
A. Bearing Wall System					
1. Special reinforced concrete shear wall	5	5	NL	NL	50
2. Ordinary reinforced concrete shear wall	4	4	NL	NL	NP
3. Ordinary reinforced masonry shear wall	2	1.75	NL	50	NP
4. Ordinary plain masonry shear wall	1.5	1.25	18	NP	NP
B. Building Frame System					
5. Special reinforced concrete shear wall	5	4.25	NL	NL	NP
6. Ordinary reinforced concrete shear wall	2	2	NL	50	NP
7. Ordinary reinforced masonry shear wall	1.5	1.25	18	NP	NP

Choice of Structural System

Seismic Force Resisting System	R	C_d	SDC-B	SDC-C	SDC-D
			Height Limit (m)		
C. Moment Resisting Frame System					
4. Special RC moment frame	8	5.5	NL	NL	NL
5. Intermediate RC moment frame	5	4.5	NL	NL	NP
6. Ordinary RC moment frame	3	2.5	NL	NP	NP
D. Dual Systems: SMF Capable of 25% V					
3. Special RC shear wall	7	5.5	NL	NL	NL
4. Ordinary RC shear wall	6	5	NL	NL	NP
E. Dual Systems: IMF Capable of 25% V					
3. Special RC shear wall	6.5	5	NL	NL	50
4. Ordinary RC shear wall	5.5	4.5	NL	NL	NP
F. Dual Systems: Ordinary RC Moment Frame and Ordinary RC Shear wall	4.5	4	NL	NP	NP

Zone Factor(Z)

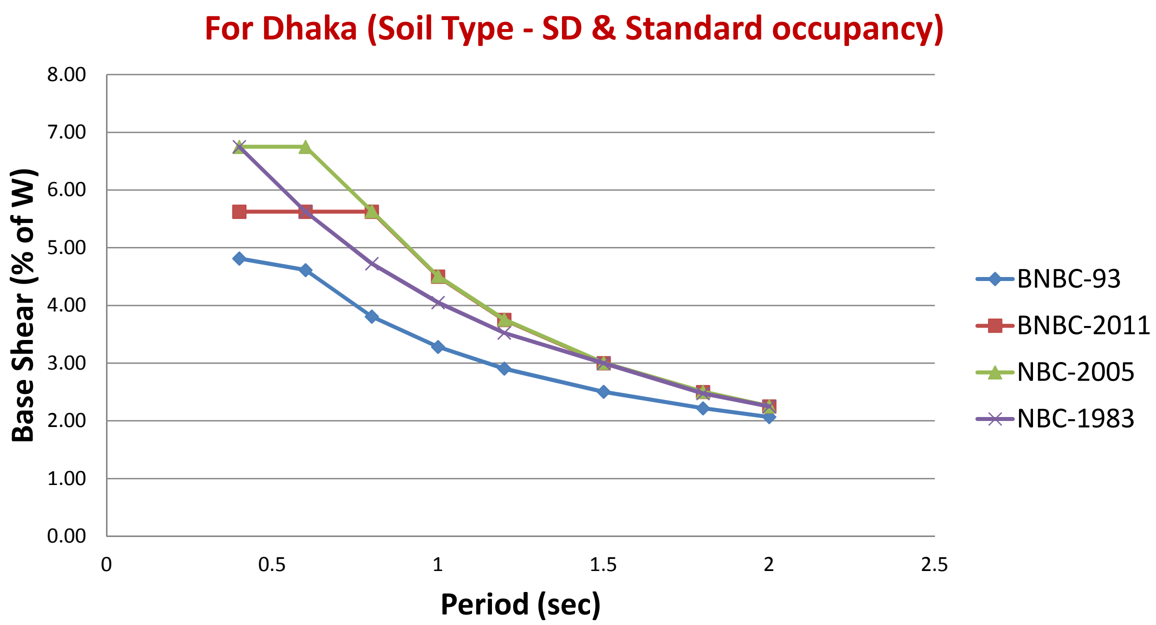


Comparison of Base Shear

For Dhaka (Soil Type - SD & Standard occupancy)

Structural period (T)	Base shear (% of wt)		% change in BNBC-2011	Base shear (% of wt) according to Indian Code	
	BNBC-93	BNBC-2011		NBC-2005	NBC-1983
0.4	4.81	5.63	16.88%	6.75	6.75
0.6	4.61	5.63	21.95%	6.75	5.63
0.8	3.81	5.63	47.73%	5.64	4.73
1	3.28	4.50	37.14%	4.51	4.05
1.2	2.91	3.75	29.06%	3.76	3.53
1.5	2.50	3.00	19.81%	3.01	3.00
1.8	2.22	2.50	12.74%	2.51	2.48
2	2.07	2.25	8.85%	2.25	2.25

Comparison of Base Shear in Various Codes



Comparison of Base Shear

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For Dhaka (Soil Type - SD & Essential occupancy)

Structural period (T)	Base shear (% of wt)		% change in BNBC-2011	Base shear (% of wt) according to Indian Code	
	BNBC-93	BNBC-2011		NBC-2005	NBC-1983
0.4	6.02	8.44	40.26%	10.13	10.13
0.6	5.77	8.44	46.34%	10.13	8.44
0.8	4.76	8.44	77.28%	8.45	7.09
1	4.10	6.75	64.57%	6.76	6.08
1.2	3.63	5.63	54.87%	5.64	5.29
1.5	3.13	4.50	43.77%	4.51	4.50
1.8	2.77	3.75	35.29%	3.76	3.71
2	2.58	3.38	30.62%	3.38	3.38

Comparison of Base Shear in Various Codes

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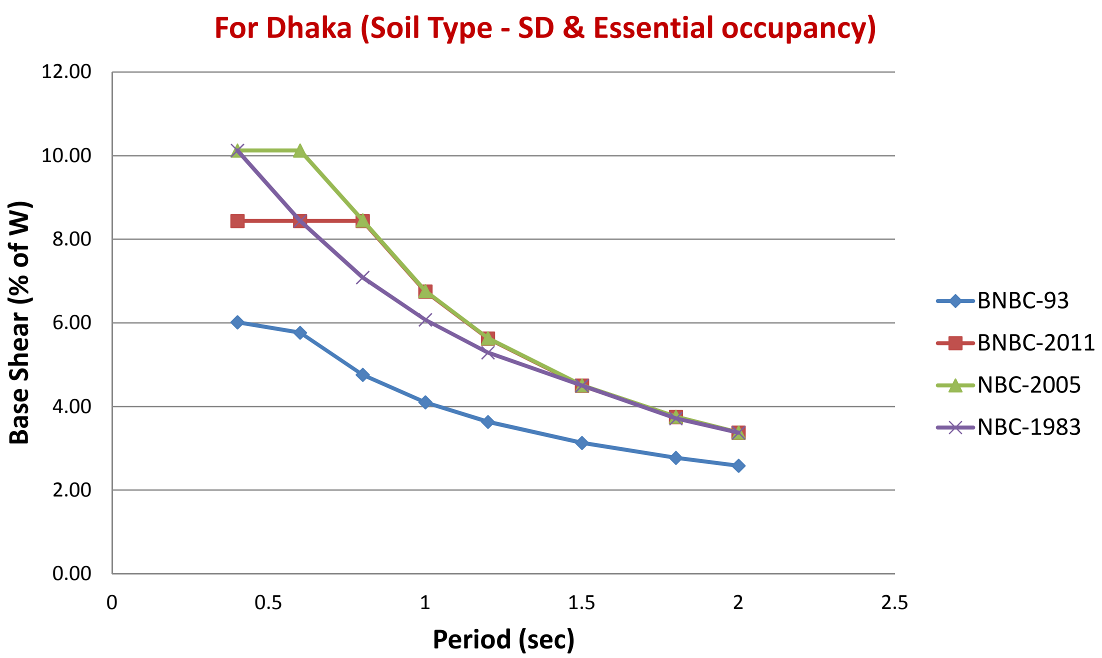

Building Period (T)

a) Structural dynamics procedure (Rayleigh method):

$$T_A = 2\pi \sqrt{\sum_{i=1}^{n} w_i \delta_i^{\,2} \Big/ g \sum_{i=1}^{n} f_i \delta_i}$$

b) Approximate method:

$$T_B = C_t (h_n)^m$$

h_n =Height of building in meter

Structure Type	C_t	m
Concrete moment resisting frames	0.0466	0.9
Steel moment resisting frames	0.0724	0.8
Eccentrically braced steel frame	0.0731	0.75
All other structural systems	0.0488	0.75

$$T_A \le 1.4 T_B$$

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Vertical distribution of EQ force

$$F_x = V \frac{w_x h_x^{\,k}}{\sum_{i=1}^{n} w_i h_i^{\,k}}$$

F_x = Part of base shear force induced at level x

w_i and w_x = Seismic weight of structure at level i and x

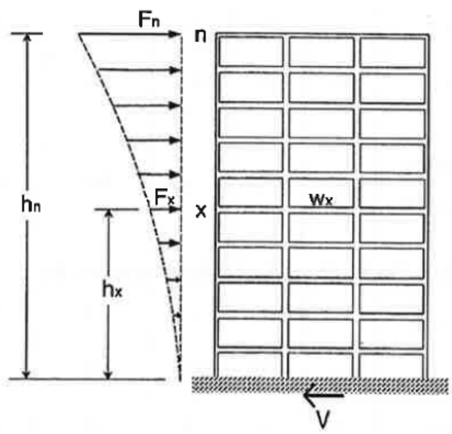
h_i and h_x = Height from base to level i and x

k = 1 for structure period ≤ 0.5 sec

= 2 for structure period ≥ 2.5 sec

= linear interpolation for other period between 1.0 and 2.0

n = number of stories



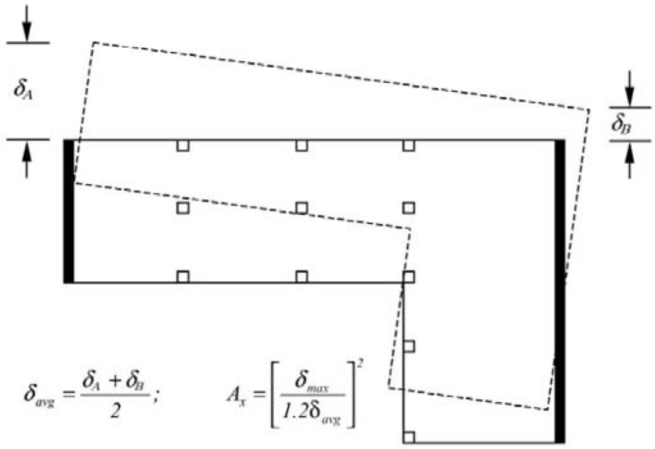
Accidental Torsional Effect

Accidental torsional moment in regular structure $M_{tai} = e_{ai} F_i$

e_{ai} = Accidental eccentricity of floor mass at level i = $\pm 0.05 L_i$

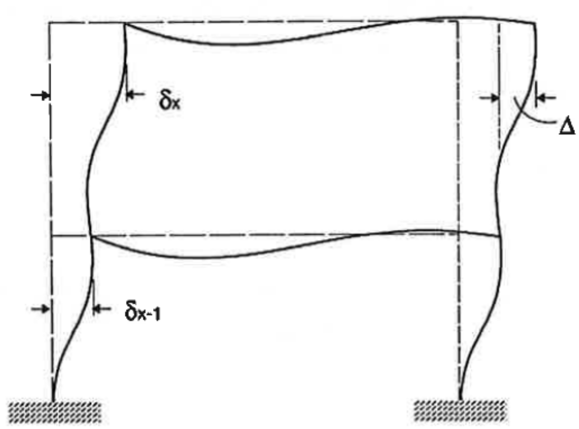
Where torsional irregularity exist in SDC-C and SDC-D increase accidental torsion, M_{ta} by A_x

$$A_x = \left[\delta_{max} / (1.2\delta_{avg})\right]^2 \leq 3.0$$



Deflection and Story Drift

Deflection at level x, $\delta_x = \frac{C_d \delta_{xe}}{I}$



C_d = Deflection amplification factor

δ_{xe} = Deflection determined by an elastic analysis

I = Importance factor

Check deflection at center of mass

Story drift at story x, $\Delta_x = \delta_x - \delta_{x-1}$

Allowable Story Drift Limit

Structure	Occupancy Category		
	I and II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

NOTES:

1. h_{sx} is the story height below Level x.
2. There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the storey drifts.
3. Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.
4. Occupancy categories are defined in Table 1.2.1

Guideline for EQ resistant Building

1. Building shall be approximately symmetrical with respect to stiffness and mass distribution.
2. Both lateral stiffness and mass of an individual story shall remain constant or reduce gradually, without abrupt change.
3. All structural elements such as cores, structural walls or frames shall run without interruption from foundation to the top.
4. An irregular building may be subdivided into dynamically independent regular unit well separated against pounding.
5. The length by breadth ratio of the building in plan shall not be more than 4.

Effects of P-Delta

P-Delta effects are not required to be considered if stability coefficient $\theta \leq 0.10$, where

$$\theta = \frac{P_X \Delta}{V_x h_{sx} C_d}$$

P_x = Vertical load above level x (with individual load factor ≤ 1.0)
Δ = Design story drift occurring simultaneously with V_x
V_x = Story shear force acting between level x and x-1
h_{sx} = Story height below level x
C_d = Deflection amplification factor

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \qquad \text{conservatively, } \beta = 1.0$$

If $0.10 \leq \theta \leq \theta_{max}$ increase displacement and member forces by rational analysis or multiply by a factor $1.0/(1-\theta)$

Requirements for Static and Dynamic Analysis

Equivalent static analysis may be applied if two conditions satisfy:

1. The building period in two main horizontal direction is smaller than both $4T_c$ and 2 sec.
2. The building does not posses any vertical irregularity.

Dynamic analysis should be performed for following buildings:

1. Regular buildings with height greater than 40m in Zones – 2, 3, 4 and greater than 90m in Zone – 1.
2. Irregular buildings with height greater than 12m in zone – 2, 3, 4 and greater than 40m in Zone – 1.

Earthquake Load Combination

Following are the guidelines for combination of earthquake load in two orthogonal direction:

1. For structures of SDC-B the design seismic forces are permitted to be applied independently in each of two orthogonal direction.
2. Structures of SDC–C and D, in addition to applying requirements for SDC–B following combinations should be satisfied:
   "±100% in X-direction ±30% in Y-direction"
   "±30% in X-direction ±100% in Y-direction"

The combination which produce most unfavourable effect, shall be considered.

Vertical Earthquake Loading

Maximum vertical ground acceleration shall be taken as 50% of expected horizontal PGA.

The vertical seismic load effect E_v may be determined as :

$$E_v = 0.5(a_h)D$$

Where,

a_h = expected horizontal peak ground acceleration for design
= (2/3)ZS

D = effect of dead load

Load Combinations for EQ force

Common load combinations:

- 1.4D
- $1.2D + 1.6L$
- $1.2D + 1.0L + 1.0E$
- $0.9D + 1.0E$

D = Dead load
L = Live load
E = Earthquake load

Provision for Soft Story

- Soft story problem is one of the major vertical irregularity.
- Commonly it happens in open parking floor.

Following two approaches are recommended –

1. Approach-1: Perform dynamic analysis considering strength and stiffness of infill wall and calculate inelastic deformations in members.
2. Approach -2:
   a) Carry out elastic earthquake analysis neglecting effect of infill wall
   b) Beam and column of soft story to be designed for 2.5 times shear and moment derived from elastic analysis.
   c) Symmetrically placed shear wall to be designed for 1.5 times lateral shear force calculated from elastic analysis

Architectural Plan view of example

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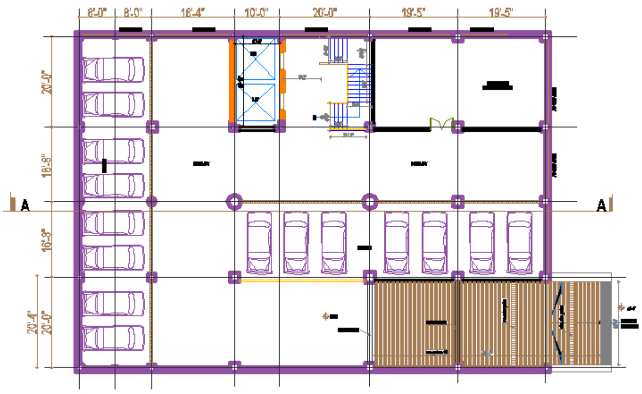
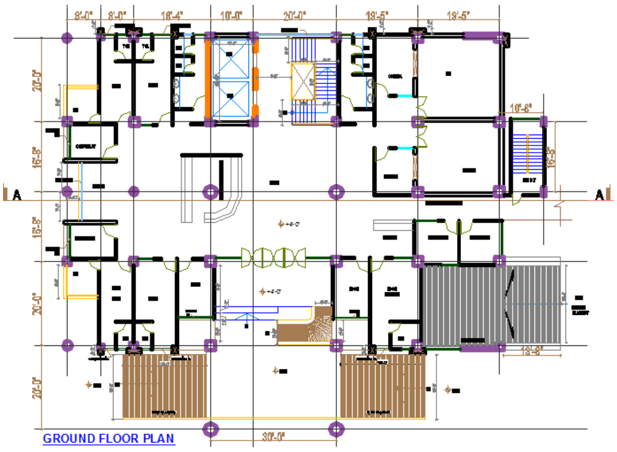


SEMI BASEMENT FLOOR PLAN



GROUND FLOOR PLAN

<u>Reference Code</u>

For Analysis:	Upcoming BNBC
For Design & Detailing	ACI 318-08

Architectural Plan view of example

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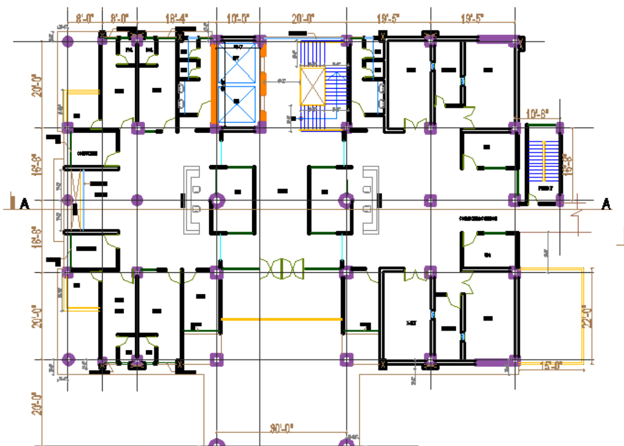
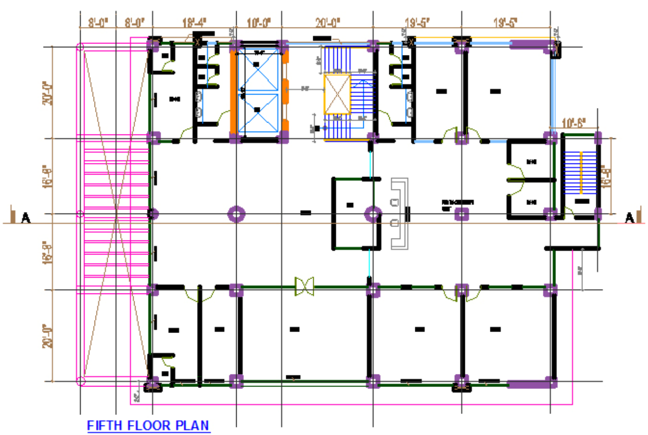


FIRST FLOOR PLAN



FIFTH FLOOR PLAN

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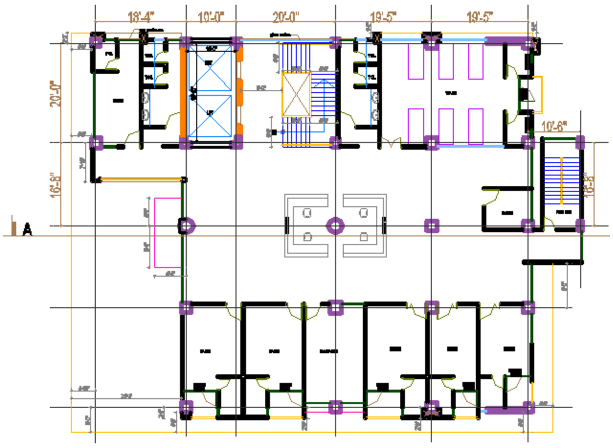
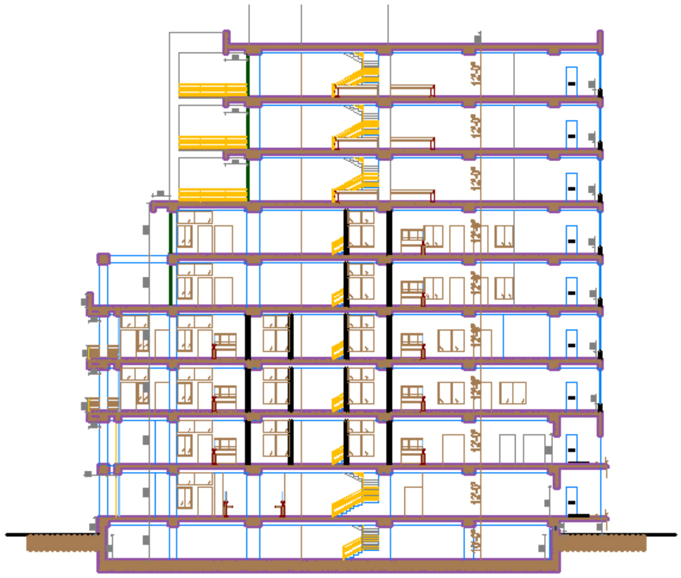
Architectural Plan view of example



SIXTH FLOOR PLAN



SECTION A-A

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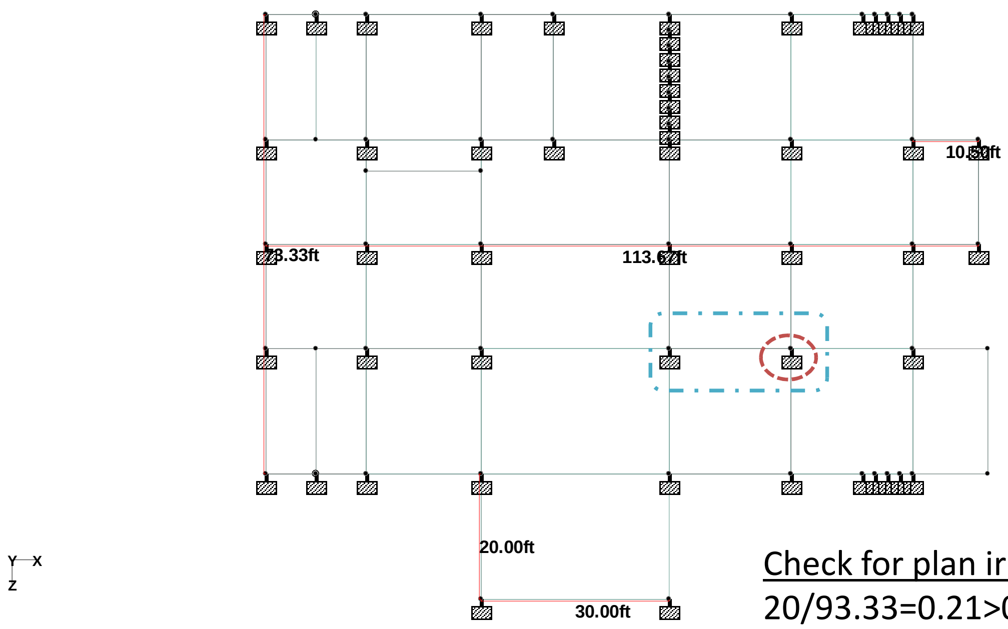
Plan view of structural model



Check for plan irregularity:
20/93.33=0.21>0.15 (Not OK)
[Ref art- 1.7.2.9(d) of BNBC 93]

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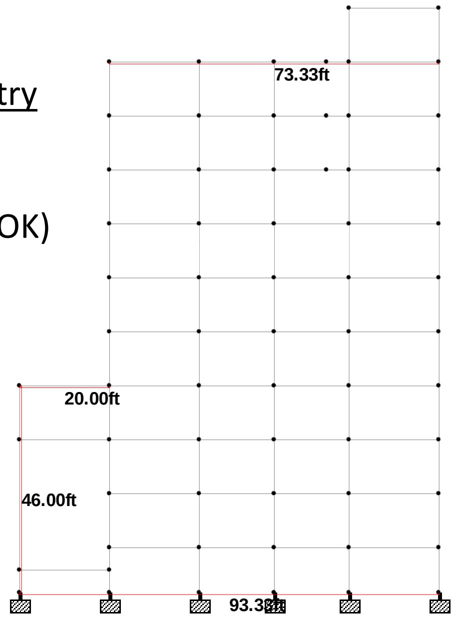
Elevation of structural model

Check for vertical geometry irregularity:

93.33/73.33=1.27<1.30 (OK)



Y
Z X

Load 1

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Selection of Seismic Design Category

1. Building located at Dhaka (Z = 0.2)
2. Hospital building (I = 1.5)
3. Soil type is SD
4. Seismic Design Category is SDC - D

Soil type	S	T_B(s)	T_C (s)	T_D (s)
SA	1.0	0.15	0.40	2.0
SB	1.2	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.4	0.15	0.50	2.0

	Occupancy Category I, II and III				Occupancy Category IV			
Site Class	Zone 1	Zone 2	Zone 3	Zone 4	Zone 1	Zone 2	Zone 3	Zone 4
SA	B	C	C	D	C	D	D	D
SB	B	C	D	D	C	D	D	D
SC	B	C	D	D	C	D	D	D
SD	C	D	D	D	D	D	D	D
SE, S_1, S_2	D	D	D	D	D	D	D	D

Selection of Structural System

Seismic Force Resisting System	R	C_d	SDC-B	SDC-C	SDC-D
			Height Limit (m)		
A. Bearing Wall System					
1. Special reinforced concrete shear wall	5	5	NL	NL	50
2. Ordinary reinforced concrete shear wall	4	4	NL	NL	NP
3. Ordinary reinforced masonry shear wall	2	1.75	NL	50	NP
4. Ordinary plain masonry shear wall	1.5	1.25	18	NP	NP
B. Building Frame System					
5. Special reinforced concrete shear wall	5	4,25	NL	NL	NP
6. Ordinary reinforced concrete shear wall	2	2	NL	50	NP
7. Ordinary reinforced masonry shear wall	1.5	1.25	18	NP	NP

Selection of Structural System

Seismic Force Resisting System	R	C_d	SDC-B	SDC-C	SDC-D
			Height Limit (m)		
C. Moment Resisting Frame System					
4. Special RC moment frame	8	5.5	NL	NL	NL
5. Intermediate RC moment frame	5	4.5	NL	NL	NP
6. Ordinary RC moment frame	3	2.5	NL	NP	NP
D. Dual Systems: SMF Capable of 25% V					
3. Special RC shear wall	7	5.5	NL	NL	NL
4. Ordinary RC shear wall	6	5	NL	NL	NP
E. Dual Systems: IMF Capable of 25% V					
2. Special RC shear wall	6.5	5	NL	NL	50
4. Ordinary RC shear wall	5.5	4.5	NL	NL	NP
F. Dual Systems: Ordinary RC Moment Frame and Ordinary RC Shear wall	4.5	4	NL	NP	NP

Calculation of Base Shear

5. Lateral load resisting system is SMF with special RC shear wall (R = 7)
6. S = 1.35, T_B = 0.2sec, T_C = 0.8 sec, T_D = 2.0 sec
7. Period of the structure is 1.08 sec.
8. $C_s = 2.5\eta(T_C/T) = 2.5$
9. $S_a = (2/3)(Z*I/R)C_s = 0.0714$
10. Minimum $S_a = (2/3)(Z*I)\beta = 0.04$
11. Weight, W = 20743 (DL) + 744 (25% of LL) = 21487 kip
12. Calculated Base Shear = S_aW = 1535 kip

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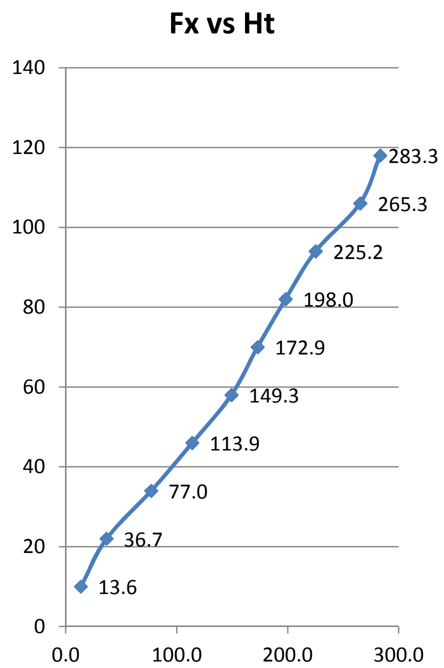
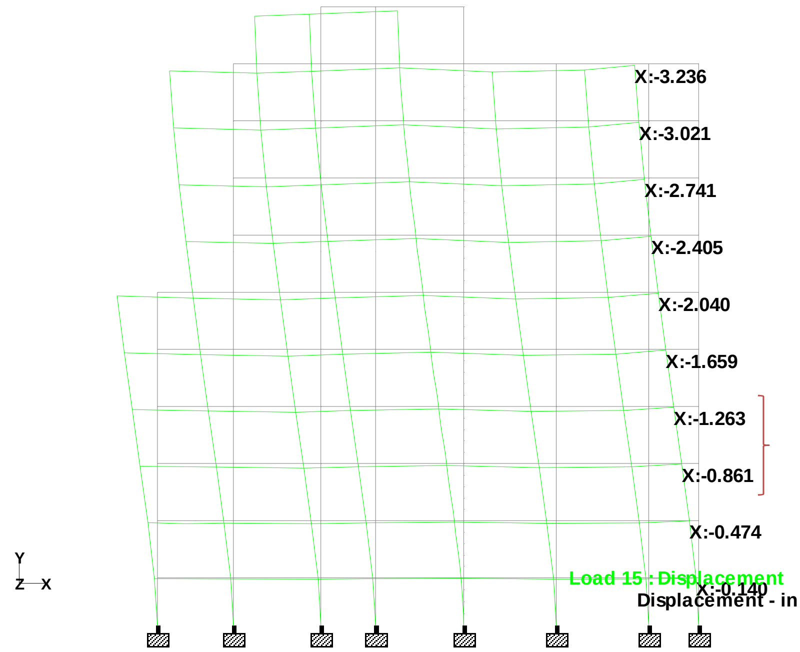
Calculation of Story Shear

K = 1.387 for T =1.08 sec

Floor level	Story weight, w_x (kip)	Height, h_x (ft)	$w_x h_x^k$	Lateral Force, F_x (kip)	Story shear, V_x (kip)
10	1677	118	1253815	283.3	283.3
9	1822	106	1173945	265.3	548.6
8	1827	94	996476	225.2	773.7
7	1942	82	876413	198.0	971.8
6	2111	70	764961	172.9	1144.6
5	2366	58	660525	149.3	1293.9
4	2491	46	504219	113.9	1407.8
3	2559	34	340590	77.0	1484.8
2	2230	22	162273	36.7	1521.4
1	2462	10	60019	13.6	1535.0
Σ	**21487**		**6793235**		



Check Storey Drift



Story drift:

δ_{xe} = 1.263 – 0.861 =0.0.402″

δ_{x} = 5.5*0.402/1.5 =1.474″

Allowable story drift

= 0.01h_{sx} =1.44” , Ok

Check for P-Delta Effect

P-Delta effects need not be considered if stability coefficient $\theta \le 0.10$

$$\theta = \frac{P_X \Delta}{V_x h_{sx} C_d}$$

At ground floor level:

P_x = 20605 kip

Δ = 1.837 inch

V_x = 1521.4 kip

h_{sx} = 12ft

C_d = 5.5

So, $\theta = 0.031 < 0.1$

$$\theta_{max} = \frac{0.5}{\beta C_d} \le 0.25$$

conservatively, $\beta = 1.0$

Here $\theta_{max} = 0.091$

If $0.10 \le \theta \le \theta_{max}$ increase displacement and member forces by rational analysis or multiply by a factor 1.0/(1- θ)

Principle for Design of SMF

- Design a strong-column/weak beam frame
- Avoid shear failure
- Detail for ductile behavior

21.1.4 – Concrete Properties of SMF

- 21.1.4.1 – Provisions apply to special moment frames, special structural walls, and coupling beams.
- 21.1.4.2 – Specified concrete compressive strength must be at least 3000 psi.
- 21.1.4.3 – Specified concrete compressive strength must not exceed 5000 psi for lightweight concrete.

21.1.5 – Reinforcement of SMF

- 21.1.5.1 – Provisions apply to special moment frames, special structural walls, and coupling beams.
- 21.1.5.2 – Deformed reinforcement must satisfy ASTM A706. Grades 40 and 60 of ASTM A 615 are permitted if:
  - The actual yield stress does not exceed the nominal yield stress by more than 18 ksi.
  - The ratio of the actual tensile strength to actual yield stress exceeds 1.25.

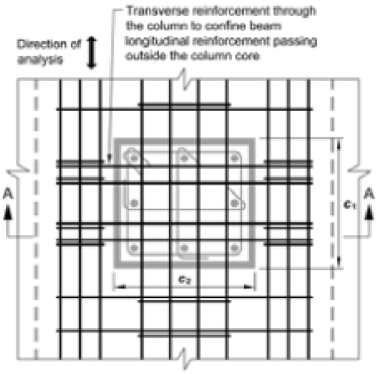
21.5 – Beams in Special Moment Frames

- A beam is defined as any frame member that resists earthquake-induced forces and is proportioned primarily to resist flexure.
- Beams must satisfy the following:
  - Factored axial compressive force must not exceed $A_g f_c'/10$.
  - Clear span must be more than 4 times the effective depth.
  - Width of member must not be less than the smaller of 0.3h and 10 in.

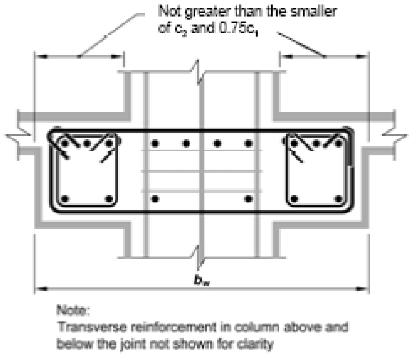
21.5 – Beams in Special Moment Frames

- 21.5.1.4 relaxed to permit wide beams.
  - $b_{w,max} = min\ (3c_2,\ c_2+1.5c_1)$

- 21.7.3.3 added to address confinement of longitudinal beam reinforcement located beyond column core.



PLAN



SECTION A-A

21.5.2 – Longitudinal Reinforcement

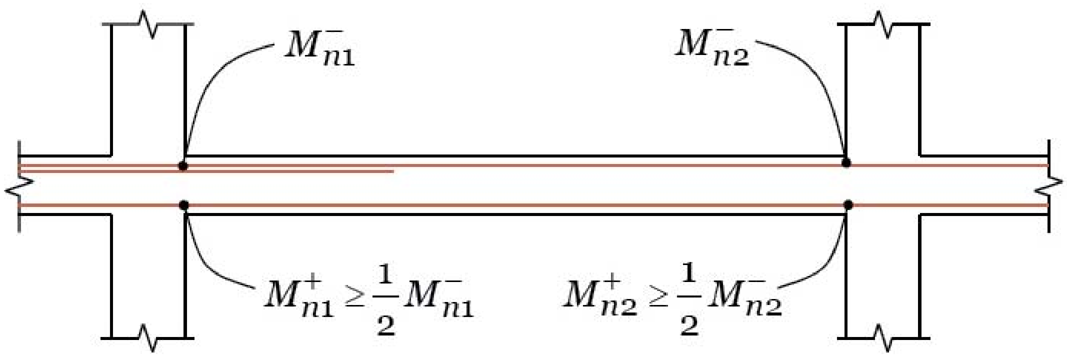
All locations:

$$A_s \geq \frac{3\sqrt{f_c'}}{f_y} b_w d \geq \frac{200}{f_y} b_w d$$

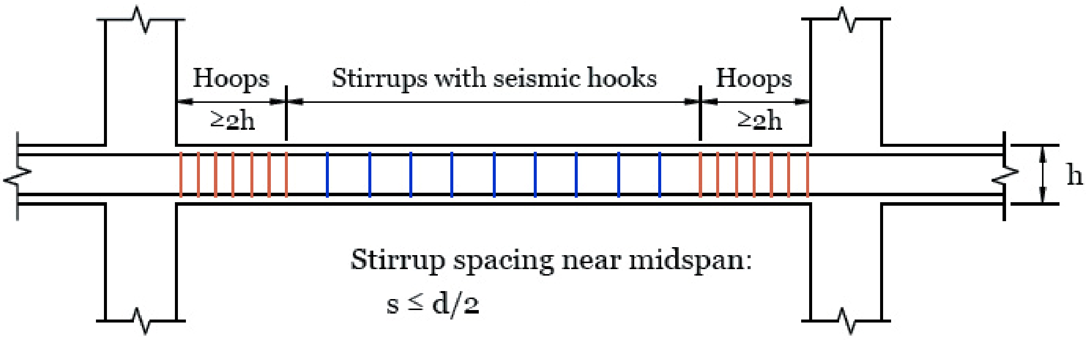
Minimum of two continuous bars per face

$$M_n \geq \frac{1}{4} \max\left(M_{n1}^-, M_{n2}^-\right)$$

$\rho \leq 0.025$



21.5.3 – Transverse Reinforceme…



Hoop spacing near joint:

$s \leq \min(d/4, 8d_b, 24d_b, 12 \text{ in.})$ = 5”, 6”, 9”, 12” for beam size 15”x24”

Hoop bar

Smallest longitudinal bar

First hoop is located no more than 2 in. from the face of the support

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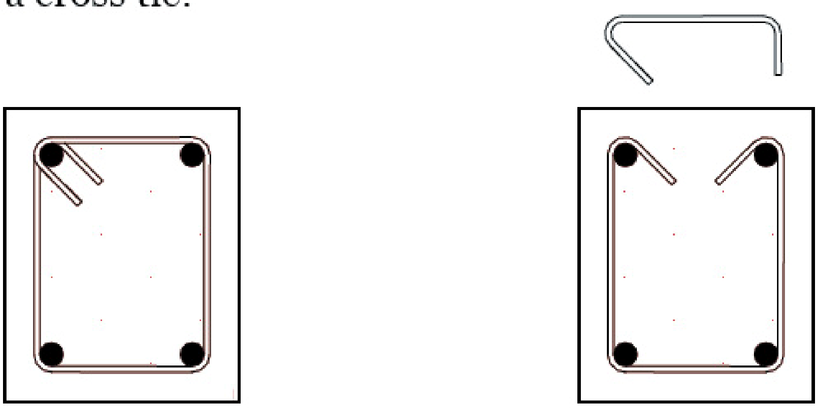
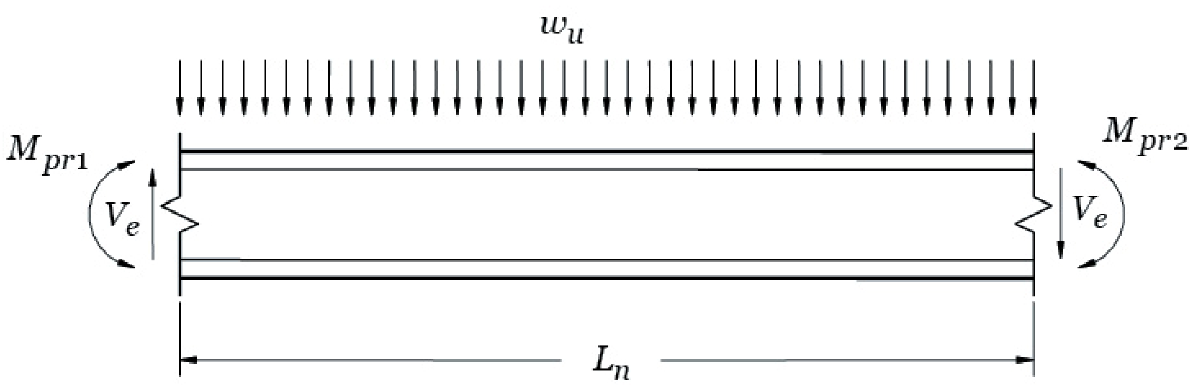
21.5.3 – Transverse Reinforceme…

- Hoops in beams are permitted to be made of two pieces of reinforcement: a stirrups having seismic hooks at both ends and a cross tie.



21.5.4 – Shear Strength Requirements



Capacity Design Approach:

$$V_e = \frac{M_{pr1} + M_{pr2}}{L_n} \pm \frac{w_u L_n}{2}$$

21.5.4 – Shear Strength Requirements

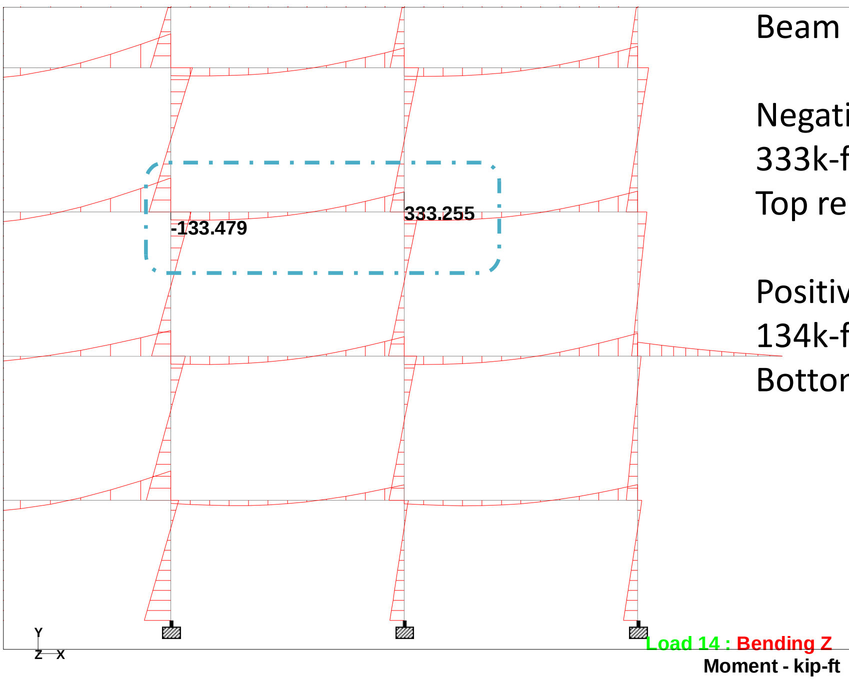
V_e = design shear force (factored shear)

M_{pr} = probable flexural strength, calculated using a stress in the reinforcement of 1.25 f_y and a strength reduction factor of 1.0.

21.5.4 – Shear Strength Requirements

- Transverse reinforcement in the regions where hoops are required shall be proportioned to resist shear assuming that
  $V_c = 0$ when both of the following conditions occur:
  - The earthquake-induced shear force represents at least 50% of the required shear strength.
  - The factored axial compressive force including earthquake effects is less than $A_g f_c'/20$.

Beam Bending moment

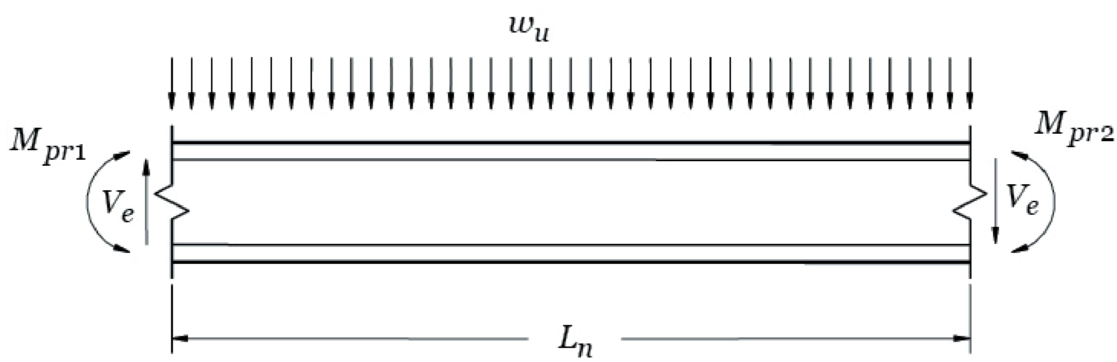


Beam size = 15” x 24”

Negative moment of beam = 333k-ft
Top rebar = 6-d25

Positive moment of beam = 134k-ft
Bottom rebar = 2-d25+2-d20

Earthquake Induced Shear Force

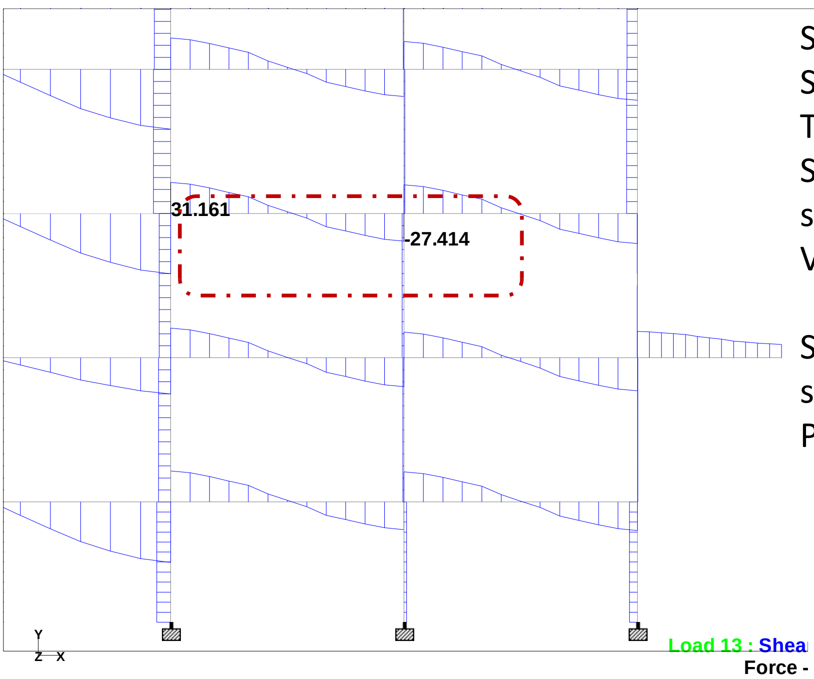


$M_{pr} = A_s * 1.25F_y * (d – a/2)$

M_{pr1} = 468 k-ft and M_{pr2} = 286 k-ft

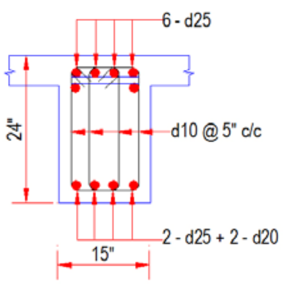
$V_{eq} = (M_{pr1} + M_{pr2})/L$ = 39.0 kip

Beam Shear Force



Shear for service load = 31.2k
Shear for EQ = 39.0k
Total shear force = 70.2k
Since 39.0>31.2 and P = 8.7 k
so Vc = 0
Vs = 70.2k

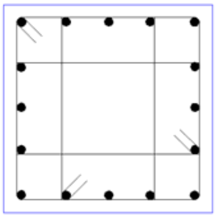
So, required hoop spacing
s = 3.37" (2leg-d10)
Provide 4-leg d10 @ 5" c/c



21.6 – Column in Special Moment Frames

- A column is defined as any frame member that resists earthquake-induced forces and has a factored axial force in any load combination that exceeds $A_g f_c'/10$.
- Columns must satisfy the following:
  - Shorter cross-sectional dimension must be at least 12 in.
  - Aspect ratio for the column must not be less than 0.4.

For axial load 1030 kip
and Moment 258k-ft
Column designed as
Size = 24”x24”
Main rebar = 16-d25
Hoop = d10 @ 4” c/c



21.6.2.2 – Strong Columns/Weak Beams

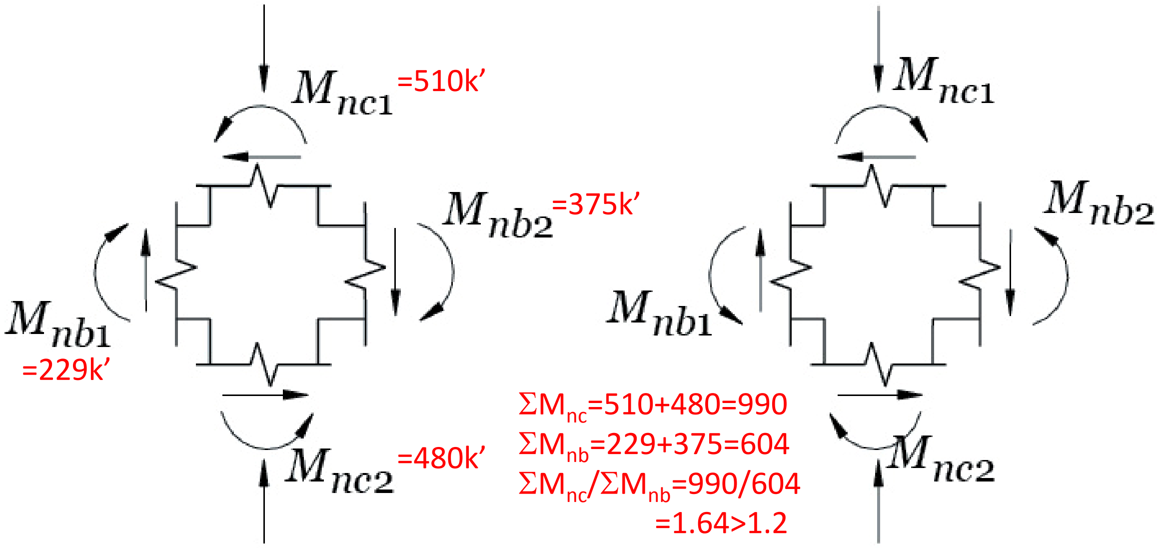
- A strong column-weak beam system must satisfy:

$$\Sigma M_{nc} \geq (6/5)\Sigma M_{nb}$$

M_{nc} = sum of moments at the faces of the joint corresponding to the nominal flexural strength of the columns framing into that joint.

M_{nb} = sum of moments at the faces of the joint corresponding to the nominal flexural strength of the girders framing into that joint. In T-beam construction, where the slab is in tension under the moments at the face of the joint, slab reinforcement within the effective slab width defined in 8.10 shall be assumed to contribute to the flexural strength if the slab reinforcement is developed at the critical section for flexure.

21.6.2.2 –
Strong Columns/Weak Beams



The nominal flexural capacities of the members are summed such that column moments oppose the beam moments. The column strengths must satisfy the relationship for beam moments acting in both directions.

21.6.2.2 –
Strong Columns/Weak Beams

- If the columns do not satisfy the requirements for strong columns, the columns must satisfy the provisions in 21.13.
- In addition, the lateral strength and stiffness of columns that do not satisfy 21.6.2.2 must be ignored when calculating the strength and stiffness of the structure.

21.6.3 – Longitudinal Reinforcement

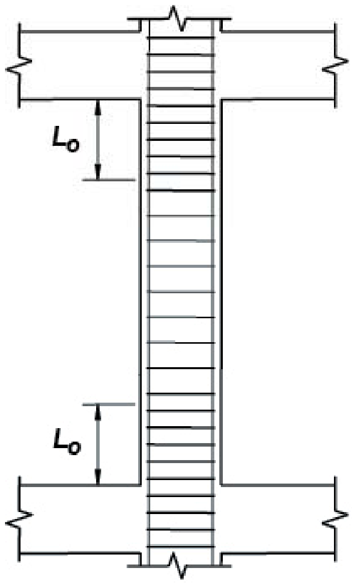
- The longitudinal reinforcement ratio must not be less than 0.01 nor more than 0.06.
- Lap splices are only permitted within the center half of the member and must be proportioned as tension splices.

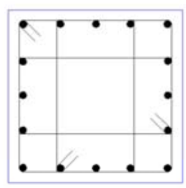
21.6.4.1 – Transverse Reinforcement

L_o is the largest of:

h, b, $\frac{L_u}{6}$, 18 in.

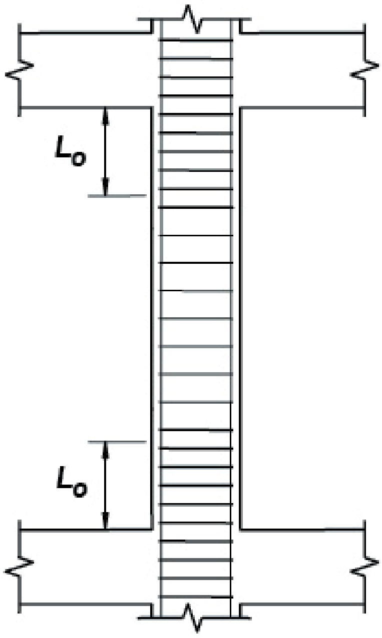
=24", 24", 20.5", 18"
for column size is 24"x 24"
And floor height 12'-0"





24" X 24"
16 - d25
d10 @ 4" c/c

21.6.4.3 – Spacing of Transverse Reinforcement



Within L_o, s must not exceed the smallest of:

$$\frac{b}{4},\ \frac{h}{4},\ 6d_b,\ s_o$$ =6", 6", 6", s (req)

For column size is 24"x 24"

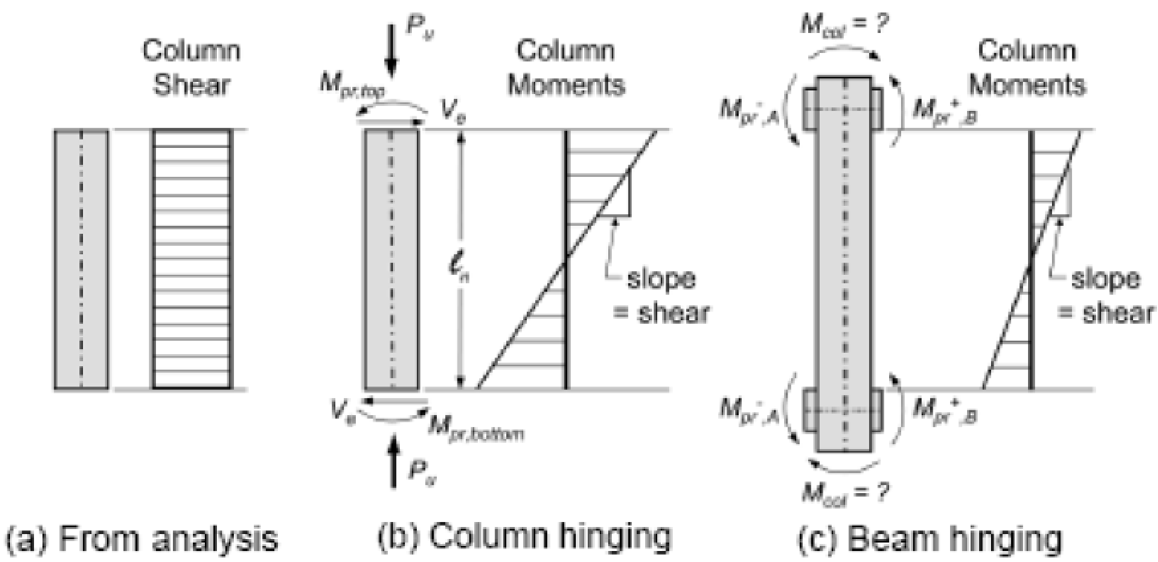
d_b = diameter of smallest longitudinal bar.

$$s_o = 4 + \left(\frac{14 - h_x}{3}\right)$$ =4.17

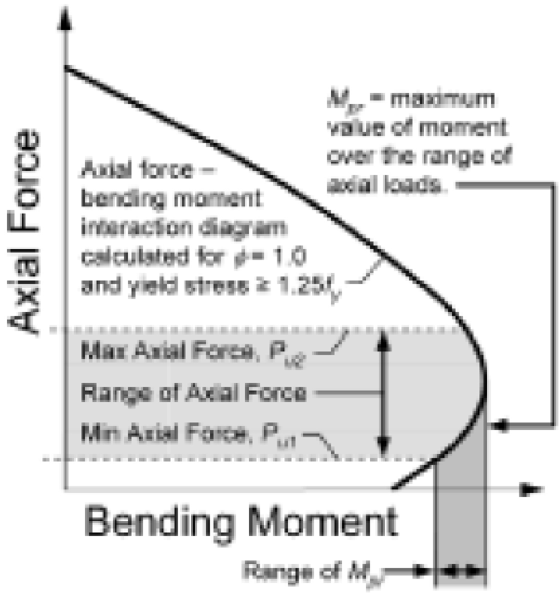
$$s_o \leq 6 \text{ in.}$$

s_o need not be taken less than 4 in.

21.6.5 – Calculation of Column Shear



Probable Moment (M_{pr}) in Column



21.6.5.2 – Shear Strength Requirements

- Transverse reinforcement over the length L_o, shall be proportioned to resist shear assuming that V_c = 0 when both of the following conditions occur:
  - The earthquake-induced shear force represents at least 50% of the required shear strength.
  - The factored axial compressive force, P_u, including earthquake effects is less than $A_g f_c'/20$.

21.6.6.4(b) – Rectangular Hoops

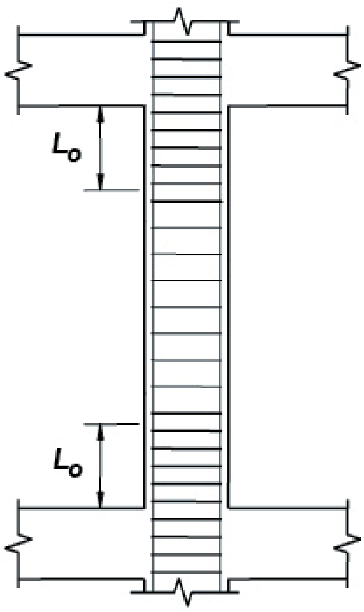
- The total cross-sectional area of rectangular hoop reinforcement must not be less than the larger of:

$$A_{sh} = 0.3\left(sb_c \frac{f'_c}{f_{yt}}\right)\left(\frac{A_g}{A_{ch}} - 1\right)$$ =0.465 < 0.48

$$A_{sh} = 0.09\left(sb_c \frac{f'_c}{f_{yt}}\right)$$ =0.456

- A_{ch} = cross-sectional area of a structural member measured to the outside edges of transverse reinforcement
- b_c = cross-sectional dimension of column core measured to the outside edges of transverse reinforcement

21.6.4.5 – Transverse Reinforcement

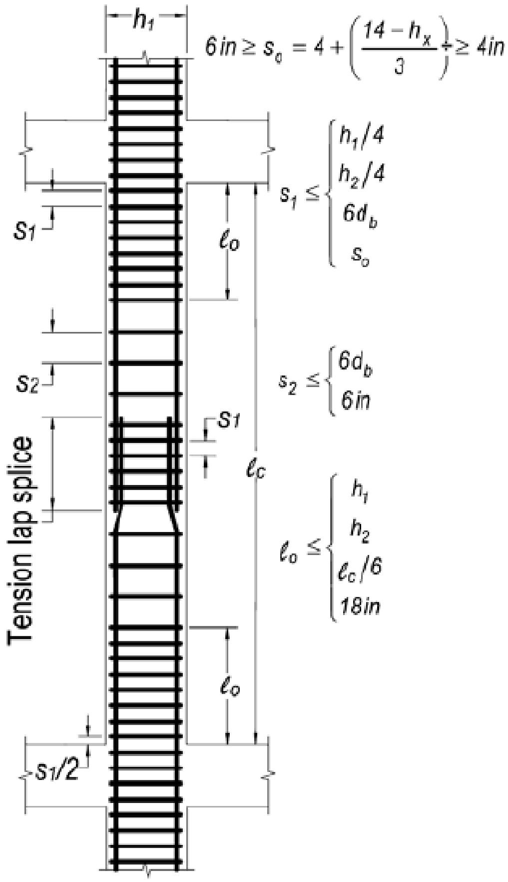


Outside L_o, the column shall contain spiral or hoop reinforcement satisfying 7.10, unless a larger amount of transverse reinforcement is required by 21.6.3.2 or 21.6.5.
s shall not exceed the smaller of:

$6d_b$, 6 in.

d_b = diameter of smallest longitudinal bar.

Typical Column Transverse Reinforcement Requirement



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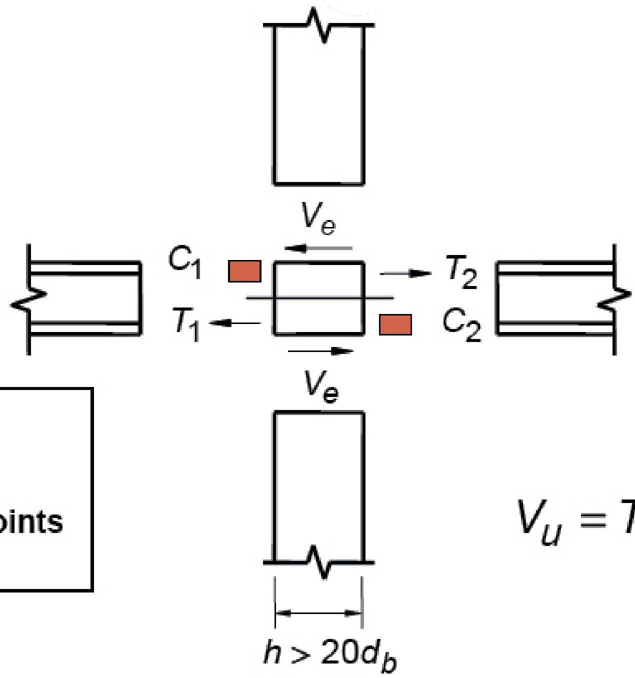
21.7 – Beam-Column Joints



$\phi = 0.85$

for shear in joints

$$V_u = T_1 + T_2 - V_e$$

21.7.2 – General Requirements

- Forces in longitudinal beam reinforcement at the joint face shall be determined by assuming that the stress in the flexural reinforcement is 1.25 fy.
- Beam reinforcement that terminates in a beam-column joint must extend to the far face of the confined core and be anchored in tension per 21.7.5 or in compression per Chapter 12.
- Where longitudinal beam reinforcement extends through a beam-column joint, the column dimensions parallel to the beam reinforcement shall not exceed 20 times the diameter of the largest longitudinal beam.

21.7.3.1~2 – Transverse Reinforcement

- The closely-spaced transverse reinforcement required near the ends of a column must be continued through the joint., except as permitted in 21.7.3.2.

- Where beams frame into all four sides of a joint and where the width of each beam is at least 75% of the column width, the amount of transverse reinforcement may be reduced by 50% and the spacing may be increased to 6 in. within the overall depth of the shallowest beam.

21.7.3.3 – Transverse Reinforcement

- Longitudinal beam reinforcement outside the column core must also be confined by transverse reinforcement that passes through the column.

- This transverse reinforcement must satisfy the spacing required by 21.5.3.2. and the requirements of 21.5.3.3 and 21.5.3.6.

21.7.4 – Shear Strength of Joint

The nominal shear strength of the joint shall not exceed the values given below:

- Joints confined on all four faces — $20\sqrt{f'_c}A_j$
- Joints confined on three faces or on two opposite faces — $15\sqrt{f'_c}A_j$
- Other joints — $12\sqrt{f'_c}A_j$

It is not possible to increase the shear strength of the joint by adding more reinforcement.

21.7.4 – Shear Strength of Joint

- A beam that frames into the face of a joint is considered to provide confinement to the joint if the area of the beam covers at least 75% of the face of the joint.

- Extensions of beams at least h beyond the joint face are considered to provide confinement. Extensions of beams must satisfy 21.5.1.3, 21.5.2.1, 21.5.3.2, 21.5.3.3, and 21.5.3.6.

21.7.4 – Shear Strength of Joint

The area of the joint, *Aj, is calculated as the joint* depth times the effective joint width.

- Joint depth is the overall depth of the column, *h.*

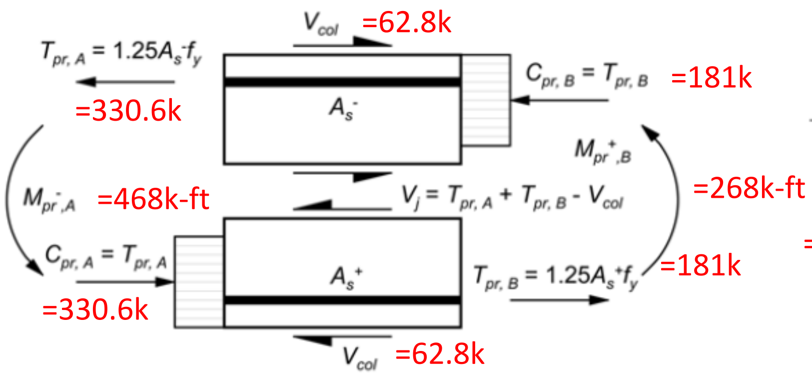
- Effective joint width is the overall width of the column, *b, except where a beam frames into a wider column.*

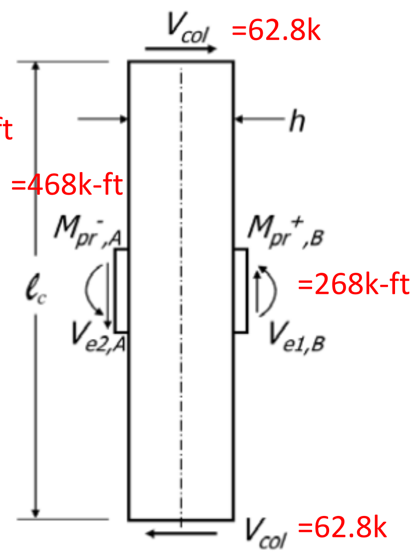
The effective joint width shall not exceed the smaller of the followings:

(a) Beam width plus joint depth.

(b) Twice the smaller perpendicular direction from the longitudinal
axis of the beam to the side of the column.

Calculation of joint strength





$$V_{col} = [(M_{pr}^{+}{}_{,B} + M_{pr}^{-}{}_{,A}) + (V_{e2,A} + V_{e1,B})\frac{h}{2}]/\ell_c$$

$M^{-}_{pr,A}$=1.25×4.56×58×(20-3)/12
=468k-ft

$T_{pr,A}$=1.25×4.56×58=330.6k

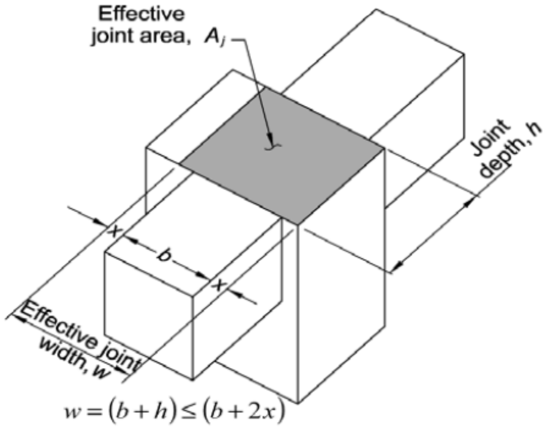
V_{col}=(468+286)/12=62.8k

Vj=330.6 + 181 – 62.8 = 448.8k

Calculation of joint strength

If beam passes through centre of the column, then
b =24” and h=24”
So, A_j = 24x24=576
φVc=12√f’c Aj
=348k < 448.8k, Not OK

If beam passes through either edges of the column, then
b =15” and h=24”
So, A_j = 15x24=360
φVc=12√f’c Aj
=216.8k < 448.8k, Not OK



21.7.5.1 – Development length of Hooked bar

- The development length for a bar with a 90° hook shall not be less than the largest of:
  - 8 $\boldsymbol{d_b}$
  - 6 in.
  - $\dfrac{f_y d_b}{65\sqrt{f'_c}}$

#3 through #11 bars
Normal weight concrete

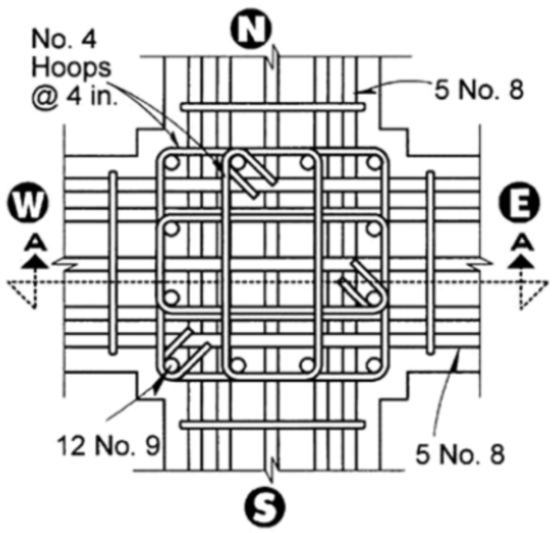
If f_y=400MPa & f'_c=3500
For 25mmϕ bar l_{dh}=15” &
For 20mmϕ bar l_{dh}=12”

- The 90° hook must be located within the confined core of a column or boundary element.

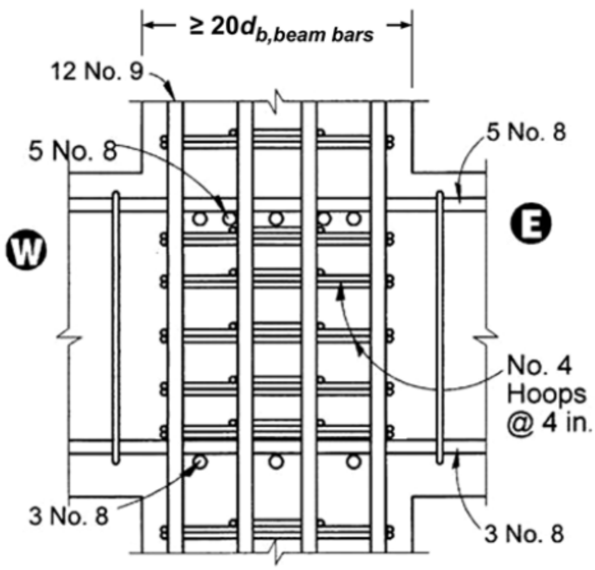
21.7.5.2 – Development length of Straight bar

- The development length of a straight bar in tension (#3 through #11) must not be less than the larger of (a) and (b):
  - (a) 2.5 times the development length for a hooked bar if the depth of concrete does not exceed 12 in.,
  - (b) 3.5 times the development length for a hooked bar if the depth of concrete exceeds 12 in.
- Straight bars terminated in a joint must pass through the confined core of a column or boundary element.
- Any portion of the straight embedded length that is not within the confined core must be increased by a factor of 1.6.

Typical Interior Joint Detailing

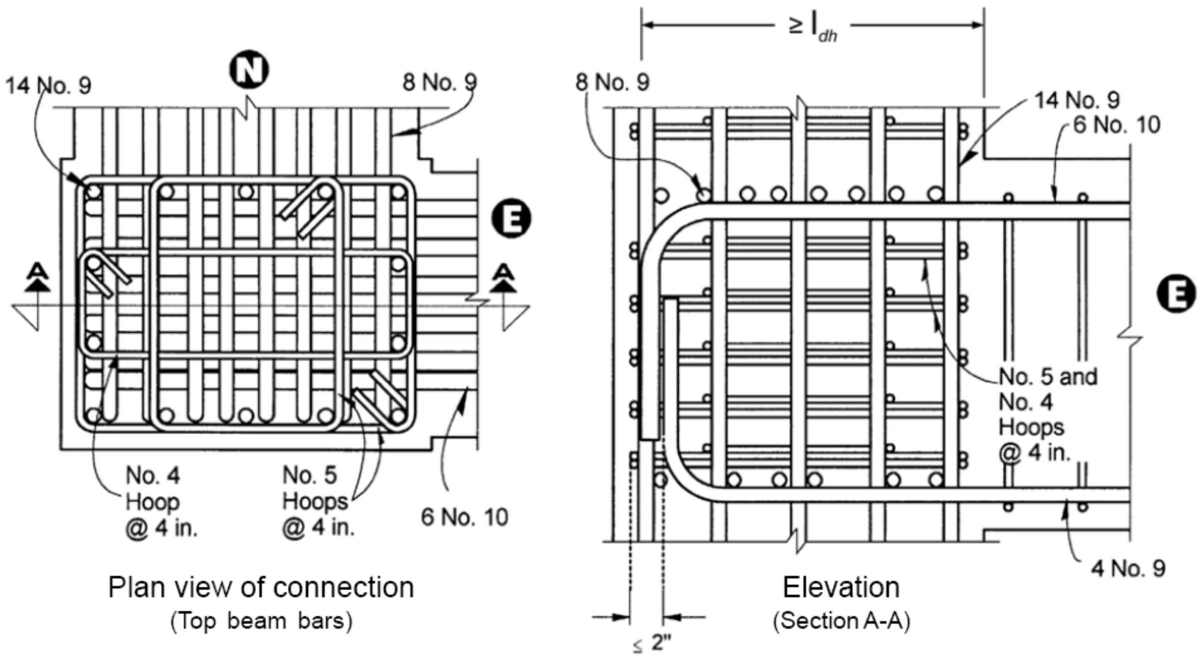


Plan view of connection
(Top beam bars)

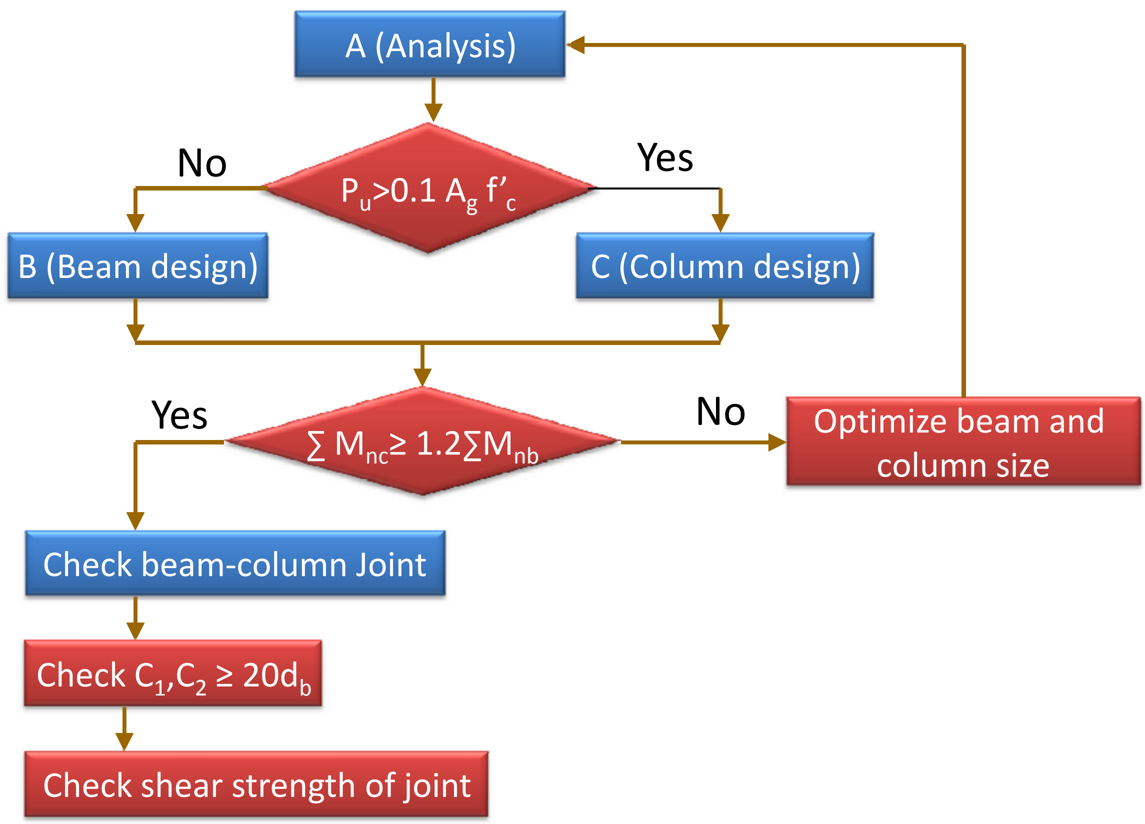


Elevation
(Section A-A)

Typical Exterior Joint Detailing



Flow Diagram for SMF (Brief)



Flow Diagram for SMF (A)

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Structural Frame type SMF
↓
Check Plan Irregularity Type –II, III, IV & V
↓
Check Vertical Irregularity Type –II, III & IV
↓
Analyze Structure with preliminary Beam and Column Size
↓
Check Story Drift
↓
Check Vertical Irregularity Type –I
↓
Check Plan Irregularity Type –I
↓
Member Design

Flow Diagram for SMF (B)

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Design member as beam
↓
Check $l_n/d \geq 4$
↓
Check $b_w \geq 0.3h$, 10”
↓
Check maximum width of beam
↓
Design beam from load combination
↓
Main rebar: $200b_wd/f_y \leq A_s \leq 0.025$
↓
Check minimum & maximum main rebar requirement
↓
Calculate hoop spacing from minimum requirement
↓
Check hoop spacing from shear strength requirement

Flow Diagram for SMF (C)

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- Design member as column
- Check $C_1/C_2 \geq 0.4$
- Check $C_1, C_2 \geq 12''$
- Design column from load combination
- Main rebar: $0.01\ A_g \leq A_s \leq 0.06\ A_g$
- Calculate hoop spacing from minimum requirement
- Calculate hoop/spiral spacing from volumetric ratio
- Calculate hoop/spiral spacing from shear strength requirement
- Check vertical irrelarity type - V

Thank you

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(CAPACITY DEVELOPMENT ON NATURAL DISASTER RESISTANT TECHNIQUES OF CONSTRUCTION AND RETROFITTING FOR PUBLIC BUILDINGS)

TECHNICAL CO-OPERATION PROJECT BETWEEN –PWD & JICA

PRESENTATION ON

EFFECTS OF INFILL BRICK MASONRY IN RC FRAME

PRESENTED BY
MD. **JAHID**UL ISLAM KHAN
MEMBER, WT-2, CNCRP

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CONTENTS

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INTRODUCTION

3

- Masonry infill walls are provided within the reinforced concrete structures without being analyzed as a combination of concrete and brick elements, though in reality they act as a single unit during earthquakes.
- The performance of such structures during earthquakes have proved to be superior in comparison to the bare frames in terms of stiffness, strength and energy dissipation.

INTRODUCTION (Cont…)

4

- There are plenty of researches done so far for infilled frames, however partially infilled frames are still the topic of interest.
- Though it has been understood that the infills play significant role in enhancing the lateral stiffness of complete structure, the past experience in various earthquakes have proved that the partially infilled framed structures somehow are affected adversely.

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CONVENTIONAL PRACTICES

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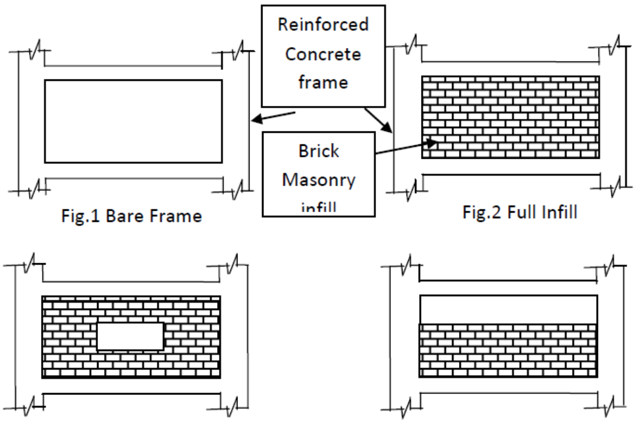

Fig.1 Bare Frame

Fig.2 Full Infill

Fig.3 Infill with opening

Fig.4 Partial Infill frame

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PROPERTIES OF INFILL WALLS

6

- If the infill walls are very light and flexible, or completely isolated from the RC frame, presence of infill does not affect the structural response of the system.
- Infill walls are expected to remain in the elastic range.
- Infill walls are expected to suffer significant damage during the seismic event.

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ADVANTAGES OF INFILL WALLS

7

- Provides durable and economical partitions.
- Higher stiffness and lower displacement.
- Higher strength.
- Lower ductility requirements for frame.
- Frame design for small lateral loads.
- Reduce contribution of frame in lateral resisting.

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DESIGN PRACTICES OF INFILL WALLS

8

- Infills are adequately separated from the RC frame such that they do not interfere with the frame under lateral deformations.
- Infills are built integral with the RC frame, but considered as non-structural elements.
- Infills are built integral with the RC frame, and considered as structural elements.

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DIFFICULTIES IN DESIGN

9

- Computational complexity.
- Structural uncertainties.
- The non-linear behaviour of infilled frames.
- Various cracking patterns and concentration of forces in structural components.

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EFFECTS OF INFILL WALLS

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- Unequal distribution of lateral forces.
- Vertical irregularities in strength and stiffness.
- Horizontal irregularities.
- Inducing the effect of short column or captive column in infilled frame.

BEHAVIOUR OF INFILL WALLS

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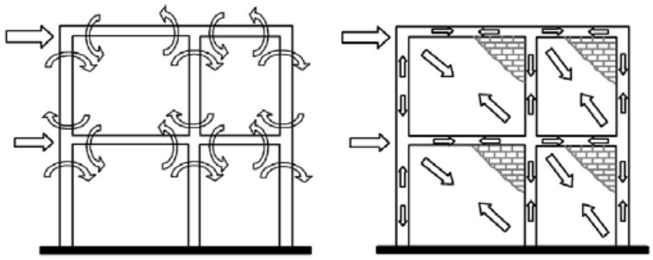
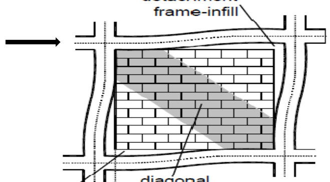
- The structural load transfer mechanism is changed from frame action to predominant truss action.



BEHAVIOUR OF INFILL WALLS (Cont…)

12

- The state of stress in the infill gives rise to a principal compressive stress along the diagonal and a principal tensile stress in the perpendicular direction.



- When infills are strong, strength contributed by the infills may be comparable to the strength of the bare frame itself.

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BEHAVIOUR OF INFILL WALLS (Cont…)

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IN-PLANE BEHAVIOUR

- The in-plane capacity of the wall depend on the relative strength of the masonry and the mortar.
- The level of the axial load significantly controls the type of failure.
- The crack propagation either follows the mortar joints or passes through the masonry units, or both.

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BEHAVIOUR OF INFILL WALLS (Cont…)

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OUT-OF-PLANE BEHAVIOUR

- Crushing along the edges for low height to thickness ratio.
- Snap-through (small effect of arching) for high height to thickness ratio i.e. approximately between 20 and 30.

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BEHAVIOUR OF INFILL WALLS (Cont…)

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PARTIALLY INFILLED FRAME

- In majority of hospitals, academic institutions and commercial complexes, partial infills are provided to attain light within the rooms.
- It is observed that such walls on one hand contribute in enhancing the lateral stiffness of the structure while on the other hand they play ironic role with an adverse effect called "short column effect".

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BEHAVIOUR OF INFILL WALLS (Cont…)

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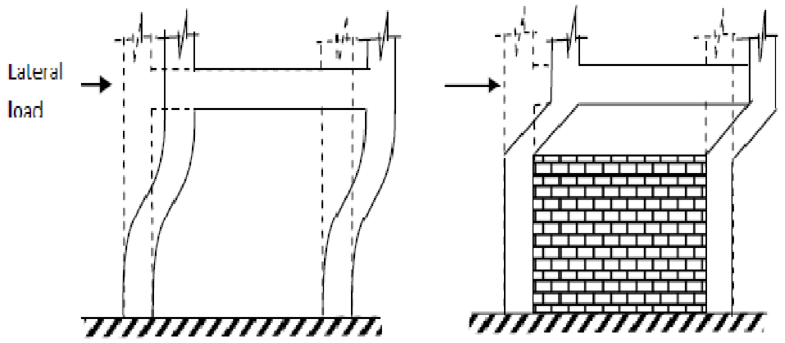
BARE FRAME

PARTIALLY INFILLED FRAME



LATERAL DEFORMATION PATTERN

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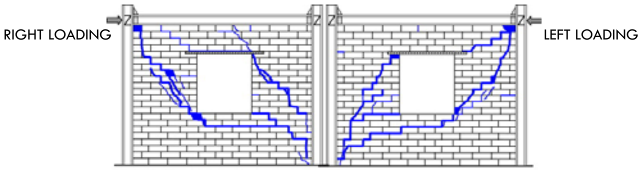
BEHAVIOUR OF INFILL WALLS (Cont…)

17

FRAMES WITH OPENNING

- In most cases, door or window openings are provided in masonry infill panels because of the functional and ventilation requirements of buildings.
- Introducing openings in an infill wall alters its behavior and adds complexity in behavior.



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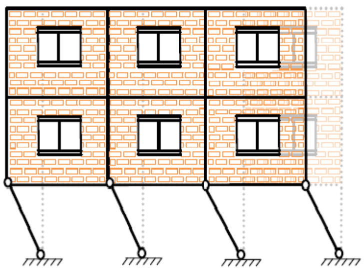
BEHAVIOUR OF INFILL WALLS (Cont…)

18

SOFT STOREY

One of the main reasons of structural failure due to earthquakes is discontinuity of lateral force resisting elements like bracing, shear wall or infill in the first storey.



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FAILURE MECHANISM

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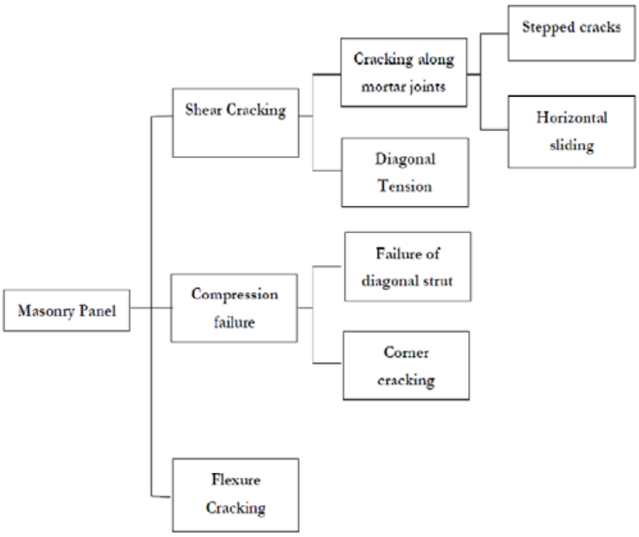

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FAILURE MECHANISM (Cont…)

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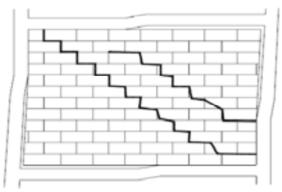
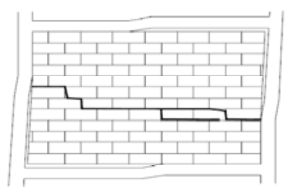
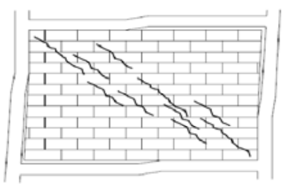
SHEAR CRACKINGS



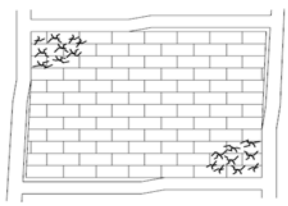
Stepped Cracking



Horizontal Sliding



Diagonal Tension



Loaded Corner

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FAILURE MECHANISM (Cont…)

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FLEXURAL CRACKING

- In those cases where flexure effects are predominating, such as multistory infilled frames, and the columns of the frame are very weak, flexure cracks can open in the tensile side of the panel due to the low tensile strength of the masonry .

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CODAL PROVISIONS-WORLD WIDE

22

- Very few design code has provisions on RC frames with brick masonry infill.
- Building code of Albania, China, India, Nepal, Eurocode, Israel, Philippines, Egypt etc.
  - have suggested dynamic analysis of structures
  - have different fundamental time period of vibration
- FEMA 306 suggested to analyze with equivalent struts and/or dynamic analysis

BNBC GUIDELINES

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- BNBC 1993 – No Design Guidelines for Frame with Infill Wall
- BNBC 2010 (Upcoming) – No Design Guidelines for Frame with Infill Wall
- BNBC 2010 (Upcoming) – There is guideline of designing soft storey in the open ground floor.

BNBC GUIDELINES (Cont…)

24

- BNBC 2010 (Upcoming) –
- 2.5.19.1
  Dynamic analysis of such building may be carried out incorpo rating the strength and stiffness of infill walls and inelastic deformations in the members, particularly those in the soft storey, and the members designed accordingly.

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BNBC GUIDELINES (Cont…)

25

- 2.5.19.2.

  Alternatively, the following design criteria are to be adopted after carrying out the earthquake analysis, neglecting the effect of infill walls in other storeys. The columns and beams of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads neglecting effect of infill walls.

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CNCRP-JICA GUIDELINES

26

- To evaluate the horizontal eccentricity and vertical stiffness of buildings, it is a very important matter that infill are considered or not considered.
- Infill walls are considerably effective to the seismic design of moderate earthquakes.
- The research for infill walls shall be more developed, and design of buildings with economy and safety shall be expected hereafter in Bangladesh.

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CNCRP-STRUCTURAL TEST

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SCALED DOWN INFILLED FRAME TEST

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FEMA GUIDELINES-EQ. STRUT METHOD

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- The effective width(s) of a diagonal compression strut that can be used to assess the stiffness and strength of an infill panel is initially calculated using the recommendations given in FEMA 306 (Chapter 8).
- The provisions are based on the early work of Mainstone (1971) and Mainstone and Weeks (1970) and are restated below for the convenience of the user.

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FEMA GUIDELINES-EQ. STRUT METHOD

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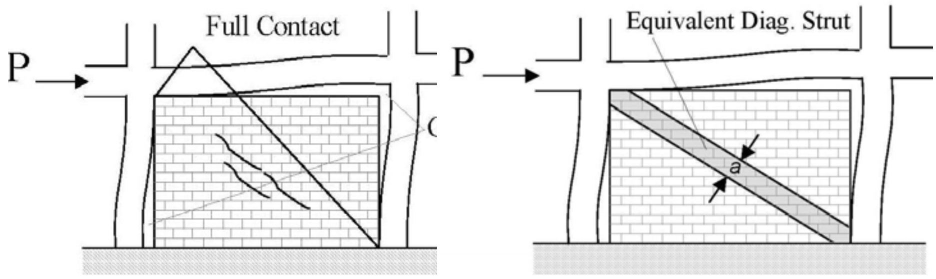

Deformation Shape

Equivalent Diagonal Strut

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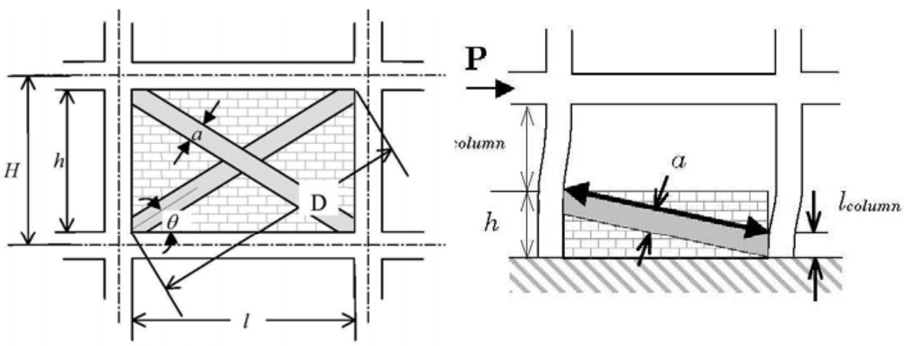
FEMA GUIDELINES-EQ. STRUT METHOD

30



Strut Geometry

Partially Infilled Frame

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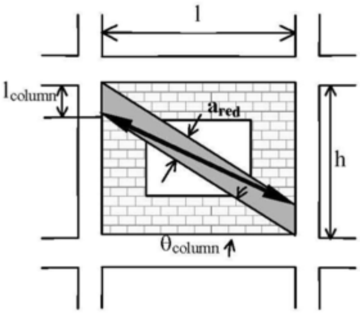
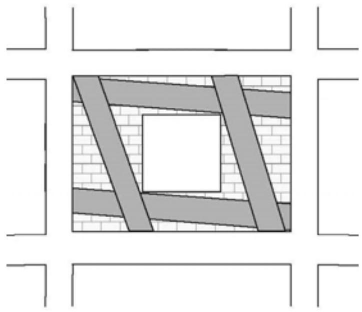
FEMA GUIDELINES-EQ. STRUT METHOD

31



Perforated Panel



Possible Strut Placement

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FEMA GUIDELINES-EQ. STRUT METHOD

32

- Thickness = Actual infill thickness (t)
- *An equivalent width, a, given by:*

$$\lambda_I H = H\left[\frac{E_m t \sin 2\theta}{4 E_c I_{col} h}\right]^{1/4}$$

By Stamford-Smith and Carter (1969)

$$a = 0.175 D(\lambda_1 H)^{-0.4}$$

By Milestone (1971)

$$a_{red} = a(R_1)_i (R_2)_i$$

$$l_{column} = \frac{a}{\cos \theta_{column}} \qquad \tan \theta_{column} = \frac{h - \dfrac{a}{\cos \theta_{column}}}{l}$$

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FEMA GUIDELINES-EQ. STRUT METHOD

33

$$R_{strut} = \min\left\{\begin{matrix} R_{cr} \\ R_{shear} / \cos\theta_{strut} \end{matrix}\right\}$$

$$R_{cr} = a_{red}\, t_{eff}\, f'_m$$

$$R_{shear} = A_n f'_v (R_1)_i (R_2)_i$$

h/t	$(R_2)_i$ for Type of Damage: Moderate	$(R_2)_i$ for Type of Damage: Severe
≤ 21	0.7	0.4
> 21	Requires Repair	

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DESIGN EXAMPLE

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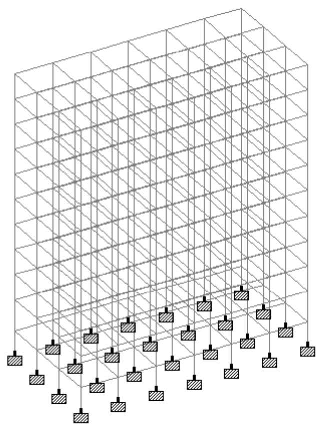
3D MODEL ANALYSIS

Wind = 260 km/hr
Seismic Zone 3
f'c = 3500 Psi, Ec = 3372 Ksi
f'm = 2000 Psi, Em = 1750 Ksi



DESIGN EXAMPLE

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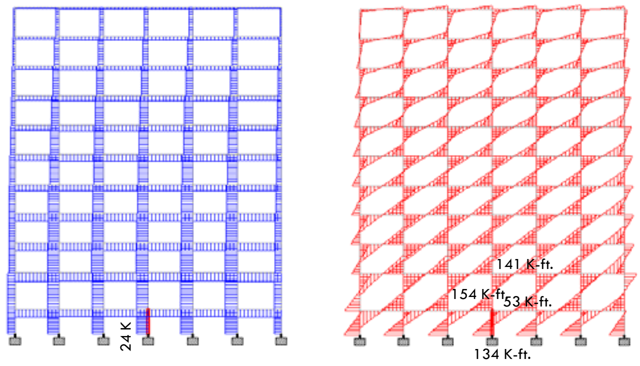
- BARE FRAME ANALYSIS



DESIGN EXAMPLE

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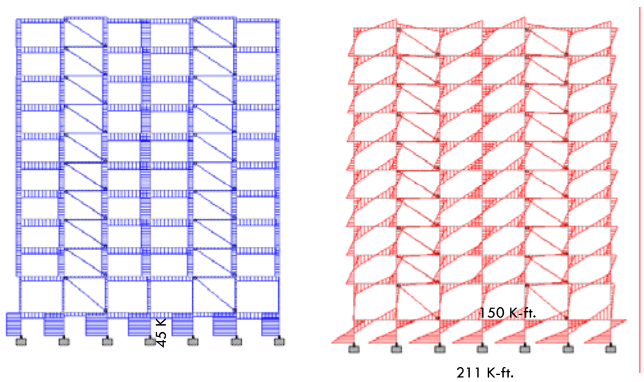
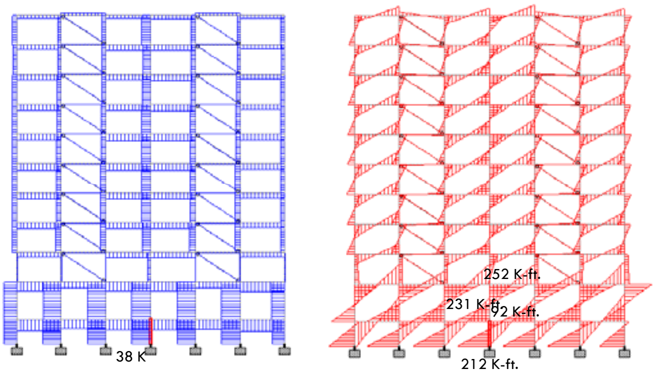
- 30% STRUTS FROM GROUND FLOOR LEVEL



DESIGN EXAMPLE

37

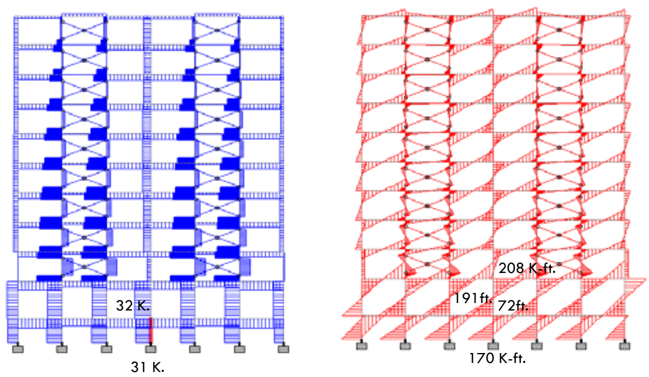
- 30% STRUTS FROM FIRST FLOOR LEVEL



DESIGN EXAMPLE

38

- 30% STRUTS AT HINGE POINTS OF COLUMNS FROM FIRST FLOOR LEVEL



CONCLUSIONS

39

- Brick infills should be considered critically during structural design to avoid horizontal and vertical irregularities of buildings
- Analysis providing equivalent struts as per FEMA 306 may be considered
- Soft storey effect should be considered and BNBC or other guidelines appropriate must be followed
- Need to approach towards dynamic analysis

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40

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- Wijanto L. S (2007) - "Seismic Assessment of Unreinforced Masonry Walls", A thesis, University of Canterbury Christchurch, New Zealand.
- Bangladesh National Building Code (BNBC 2010) - Upcoming

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THANK YOU

3rd Domestic Training

Retrofitting Construction and Quality Control

Venue: PWD Seminar Room

Duration: Feb 11, 2013 to Feb 19, 2013

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34.	MD. QUTUB AL-HOSSAIN	SUB-DIVISIONAL ENGINEER PUBLIC WORKS DEPARTMENT
35.	RIPON KUMER ROY	SUB-DIVISIONAL ENGINEER PUBLIC WORKS DEPARTMENT
36.	LT. COL. MOHIUDDIN AHMED, PEng.	INSTRUCTOR MIST

Course Title: Short Training Course on "Techniques of Retrofit Construction and Quality Control for R.C. Buildings"

Organizer: CNCRP

1. Course Duration: 3 working days (25th- 27th February, 2014)
   Lectures 2 days (2:30pm-5:30pm)
   Site visit 1 day
2. Participants: a) Engineers of PWD and other government departments
   b) Engineers from private consulting and construction firms
   (Total 36 participant/batch.)

Course content:

1. Introduction to Seismic Retrofitting
2. Seismic Assessment and Retrofit Design
3. Quality Control of Construction
4. Management of Retrofit works
5. Site visit

Course content:

<table>
<tr><td>Date</td><td>Lecture-1
(2:30-3:20)</td><td>Lecture-2
(3:20-4:10)</td><td>Break</td><td>Lecture-3
(4:30- 5:20)</td></tr>
<tr><td>25/02/14</td><td>Intro. to Seismic Retrofitting :PMT</td><td>Seismic Assessment & Retrofit Design :WT2</td><td rowspan="2">4:10 – 4:30</td><td>Retro. Methods :WT-3</td></tr>
<tr><td>26/02/14</td><td>Retro. Methods(cont.) :WT-3</td><td>Quality Control of Construction Work :WT-4</td><td>Management of Retrofit Construction:WT-3, PMT</td></tr>
<tr><td>27/02/14</td><td colspan="2">Test site at PWD premise and one government construction site visit</td><td colspan="2">Certificate Awarding</td></tr>
</table>

Retrofitting Works

Md. Sohel Rahman

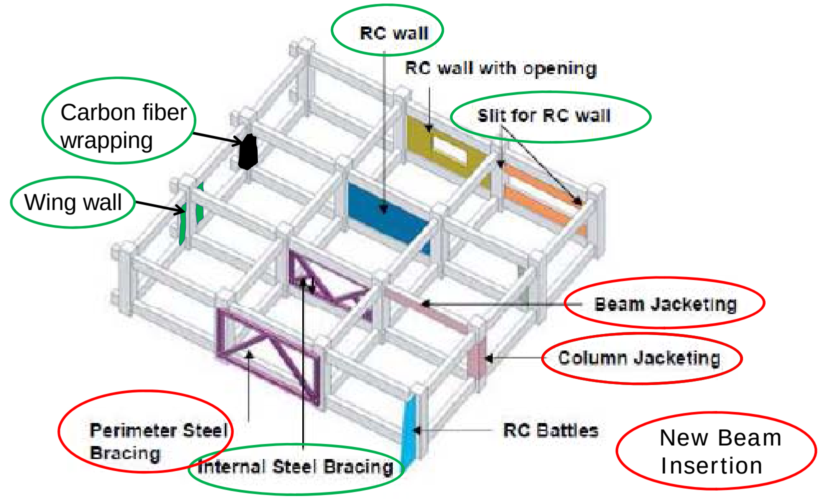
Md. Shafiul Islam

Nur-E-Kawonine

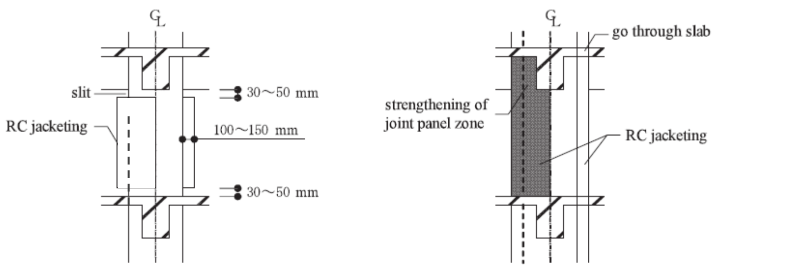
Working Team 3

CNCRP Project

Methods of Retrofitting



Retrofitting with Column Jacketing

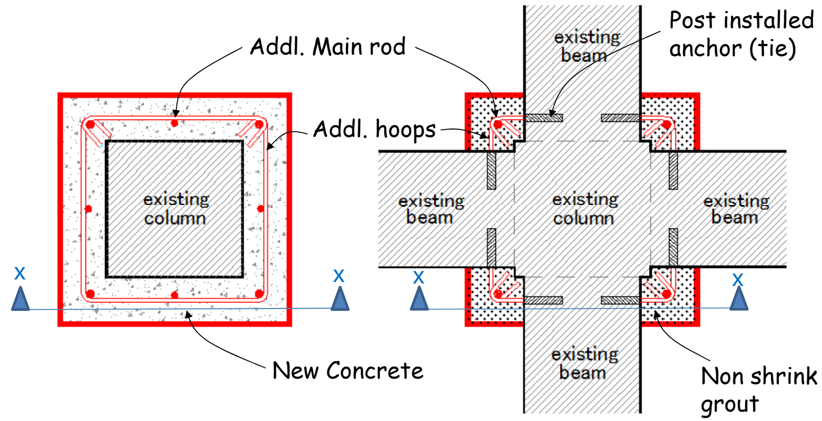


(a) in case of increase in shear strength (b) in case of increase in flexural, shear and axial strength

Figure TN.16 Column strengthening with RC jacketing

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

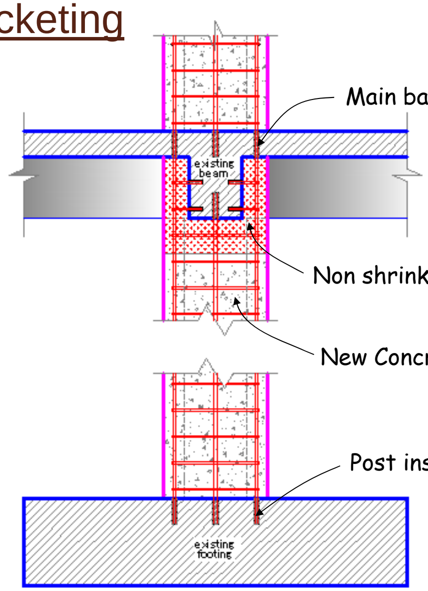
Retrofitting with Column Jacketing



Section of column at mid span

Section of column at beam-col joint

Retrofitting with Column Jacketing



Section of column jacketing

Retrofitting with Column Jacketing

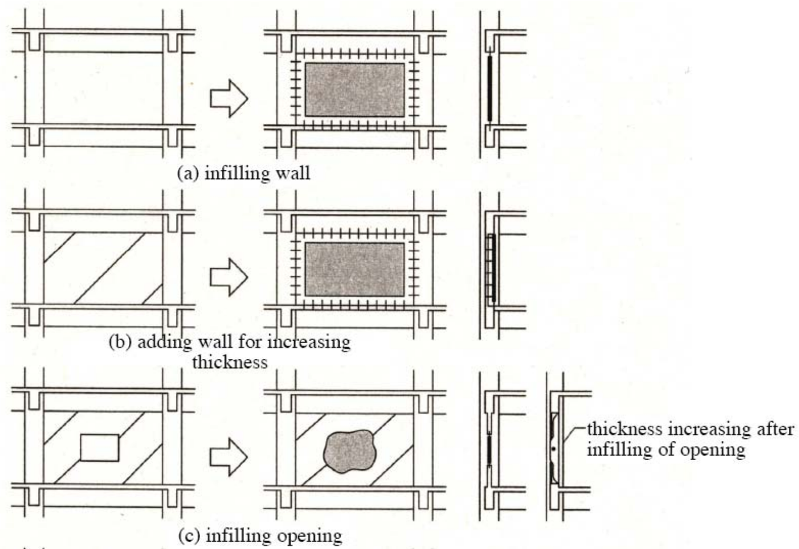
Test Work of CNCRP in 2012

Test Work of CNCRP in 2013

Retrofitting with Column Jacketing

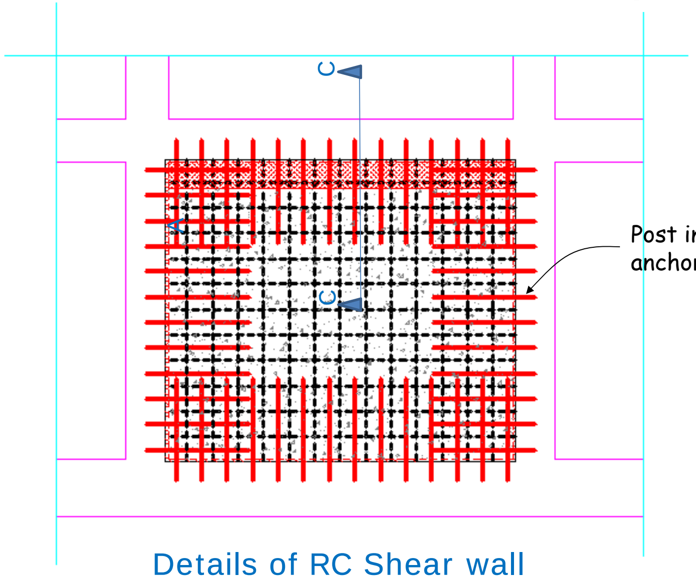
- Improve shear strength and ductility of existing building.
- To improve bonding between old and new concrete provide shear key and/ or epoxy coating over existing column.

Retrofitting with Shear Wall



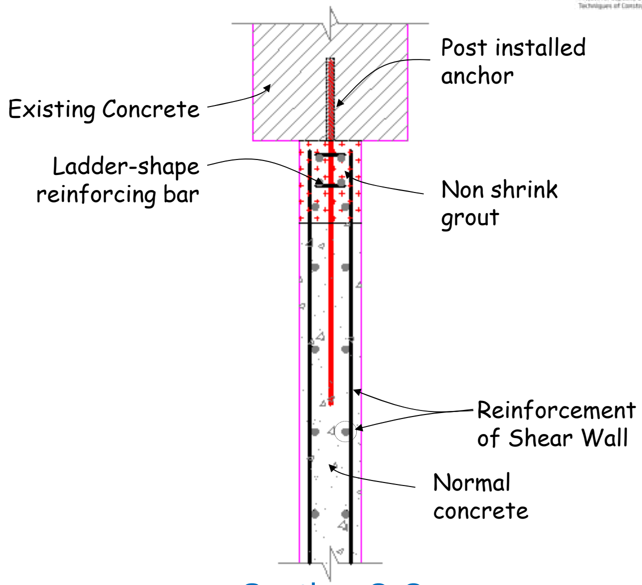
SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Retrofitting with Shear Wall



Details of RC Shear wall

Retrofitting with Shear Wall



Section C-C

Retrofitting with Shear Wall

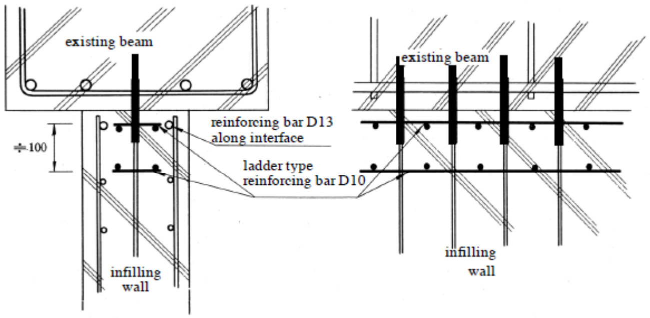


Figure TN.12 Strengthening against splitting with ladder type reinforcing bars

(quoted from the figure on page 98 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Retrofitting with Shear Wall

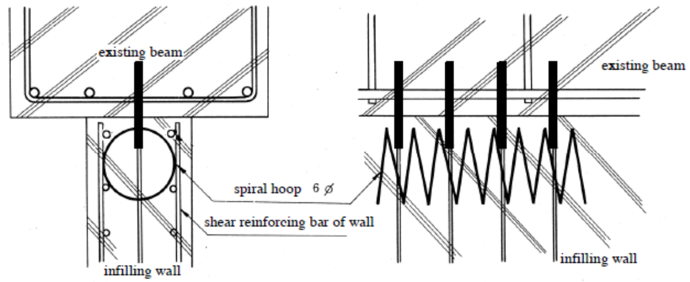
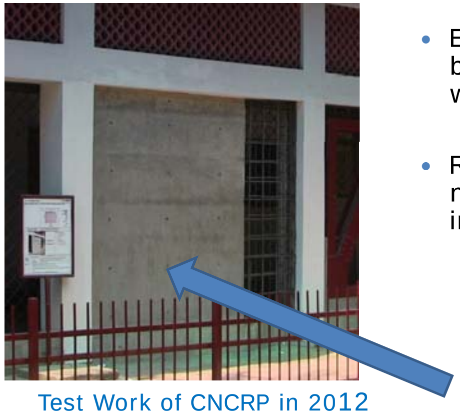


Figure TN.11 Strengthening against splitting with spiral reinforcing bars

(quoted from the figure on page 98 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Retrofitting with Shear Wall

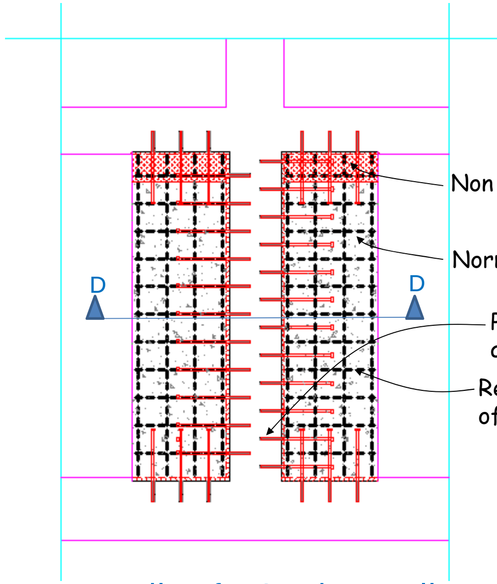


Test Work of CNCRP in 2012

- Improve shear strength and structural balance.
- Existing brick wall may be replaced by RC shear wall.
- Reduces ventilation and natural light if it is used in open frame.

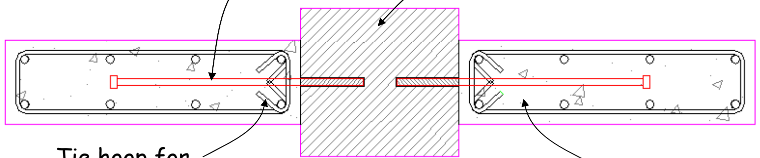
Providing RC Shear Wall in a open frame

Retrofitting with Wing Wall



Details of RC Wing wall

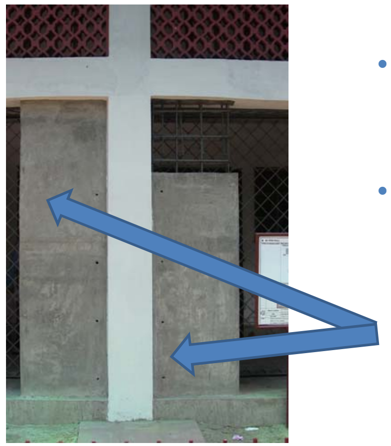
Retrofitting with Wing Wall



Section D-D

Retrofitting with Wing Wall

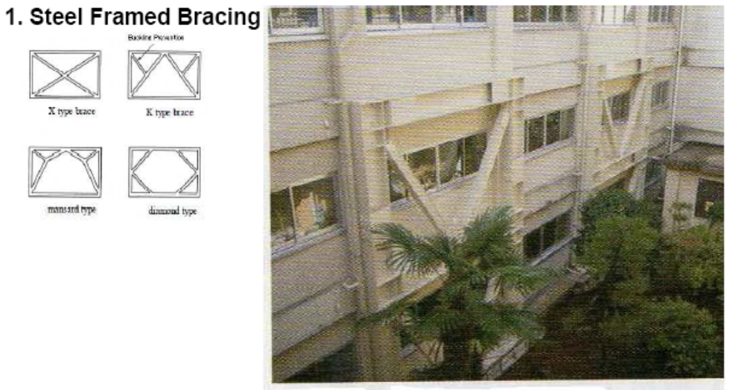
- Improve shear strength and ductility.
- failure mechanism from column yielding to beam yielding.
- Not suitable for short span beam.



RC Wing Wall Provided at an existing column

Test Work of CNCRP in 2012

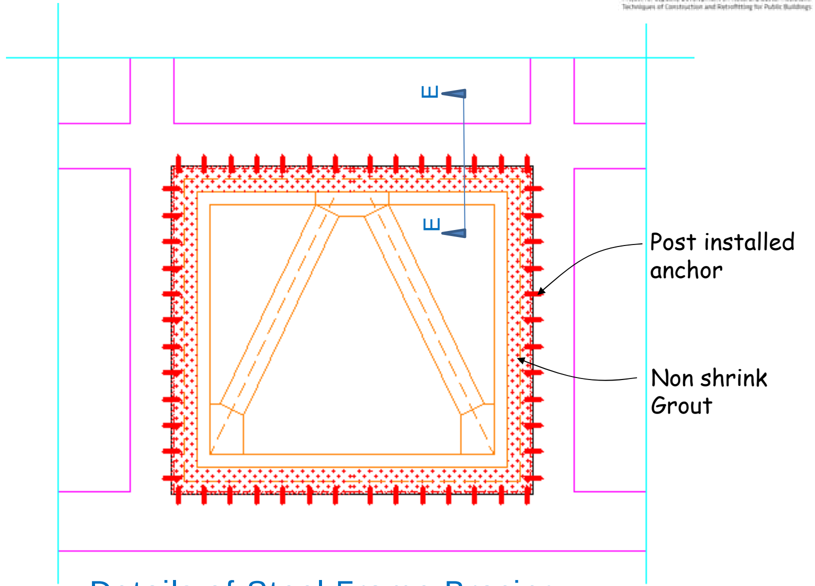
Retrofitting with Steel Frame



Ookashiwa Elementary School

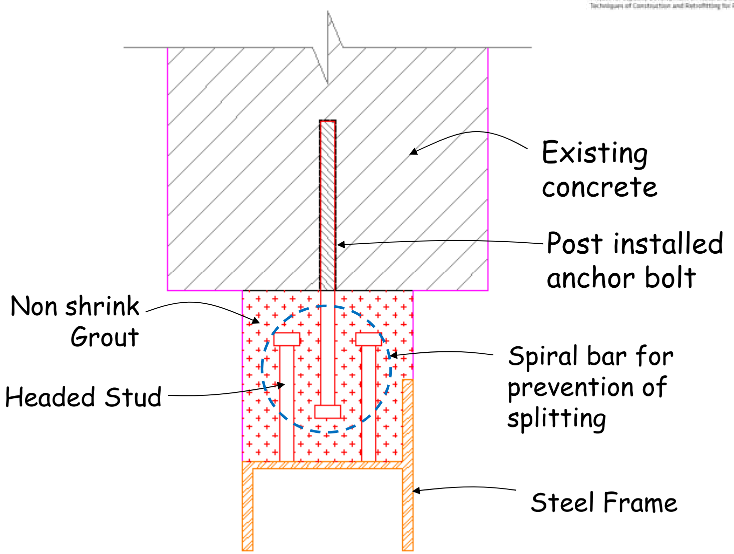
SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Retrofitting with Steel Frame



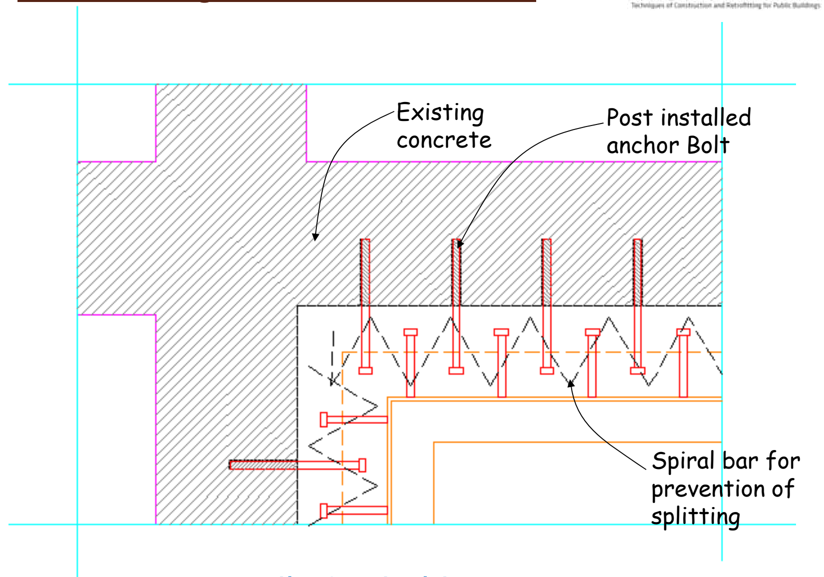
Details of Steel Frame Bracing

Retrofitting with Steel Frame



Section E-E

Retrofitting with Steel Frame



Detail of spiral bar

Retrofitting with Steel Frame

Internal Steel Frame Bracing

Test Work of CNCRP in 2012

External Steel Frame Bracing

Connection Details, Test Work 2012

Test Work of CNCRP in 2013

Steel framed bracing

a)Junior High School Building (School Colored)

b) Junior High School Building (Designed by Students)

c)University Building (with Exterior Panels)

d)Elementary School Building (with Balconies)

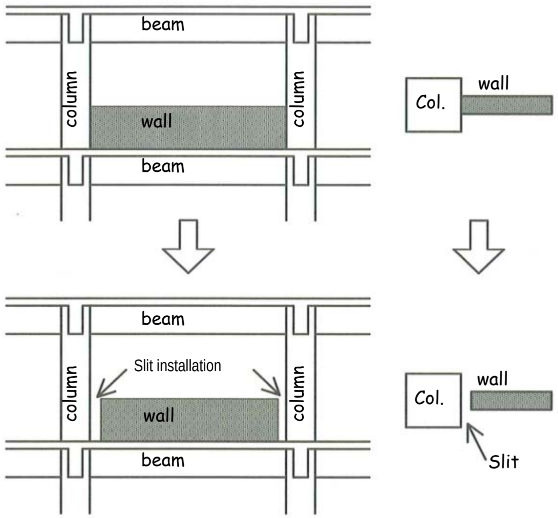
SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Retrofitting with Steel Frame

Test Work of CNCRP in 2012

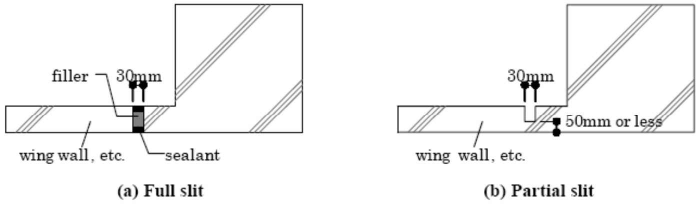
- Improve shear strength.
- soft storey/ weak storey problem may be resolved.
- Lighting and ventilation is not much disturbed, so suitable for outer frames.

Retrofitting with Structural Slit



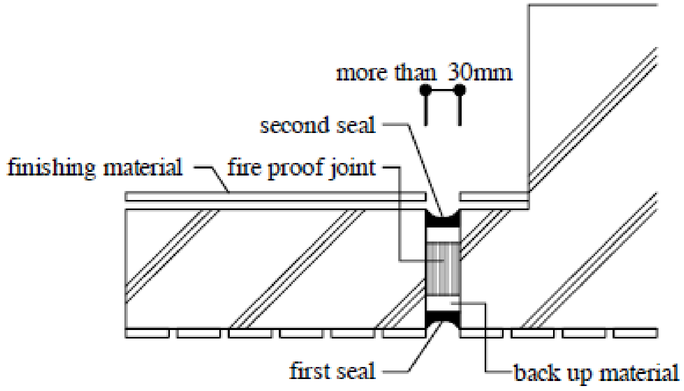
SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Retrofitting with Structural Slit



SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Retrofitting with Structural Slit



Detail of Seismic Slit

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

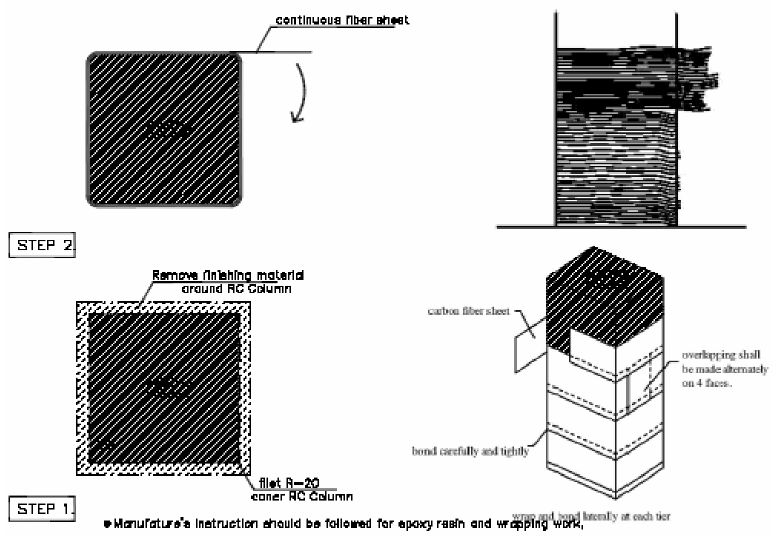
Retrofitting with Structural Slit

Seismic Slit is provided at a brick wall

Test Work of CNCRP in 2012

- To improve structural balance and ductility of existing building
- Short column failure may be avoided.

Carbon fiber sheet wrapping



Carbon fiber sheet wrapping

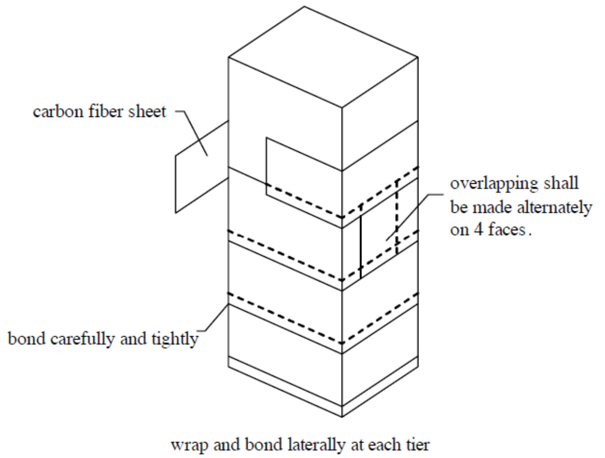


Figure TN.28 Strengthening with carbon fiber sheet wrapping

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Carbon fiber sheet wrapping

Test Work of CNCRP in 2012

Test Work of CNCRP in 2012

Carbon fiber sheet wrapping

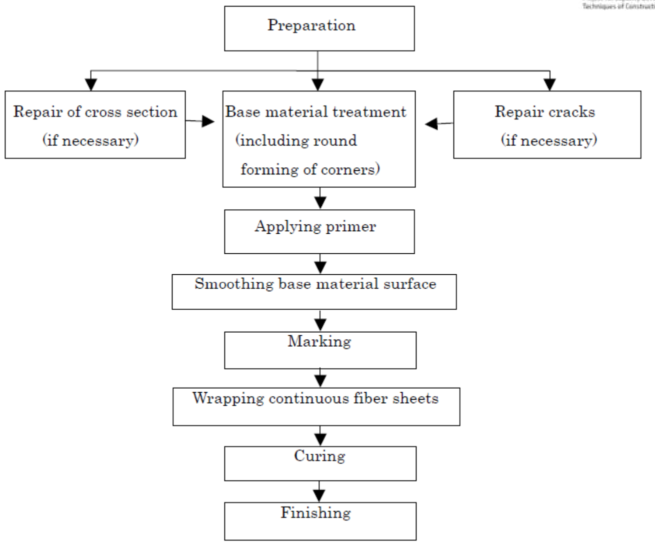


Figure 4.9-1 Flow of standard construction procedure

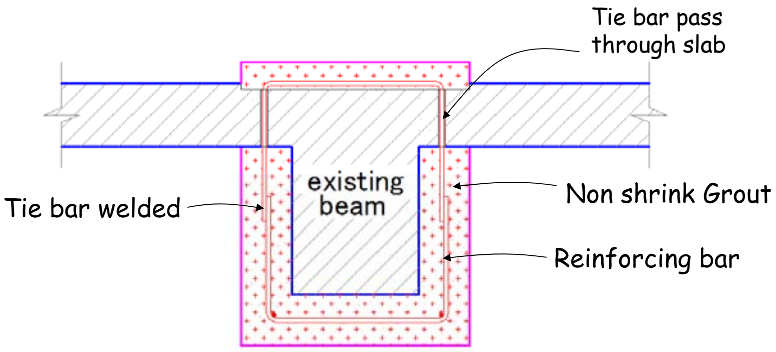
SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Carbon fiber sheet wrapping

Test Work of CNCRP in 2012

- Improve ductility.
- Skilled worker is mandatory to ensure quality construction.
- This method may be applicable on the frames with inadequate seismic hoops or tie

Retrofitting with Beam Jacketing



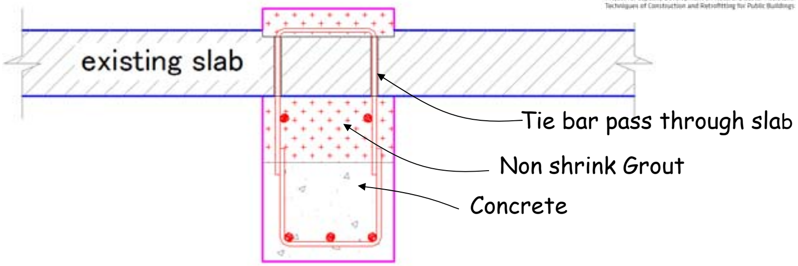
Typical Detail of Beam Jacketing

Retrofitting with Beam Jacketing

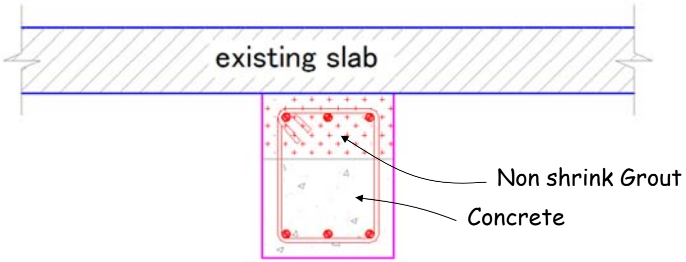
Test Work of CNCRP in 2013

- Improve ductility.

Retrofitting with Beam Insertion



Typical Detail of Beam Insertion (Option-1)



Typical Detail of Beam Insertion (Option-2)

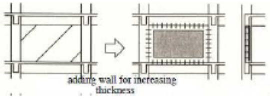
Retrofitting with Beam Insertion

Test Work of CNCRP in 2013

- Improve ductility and stiffness of the existing building.

Methods of Retrofitting

3. Increasing Thickness of Existing Shear Wall

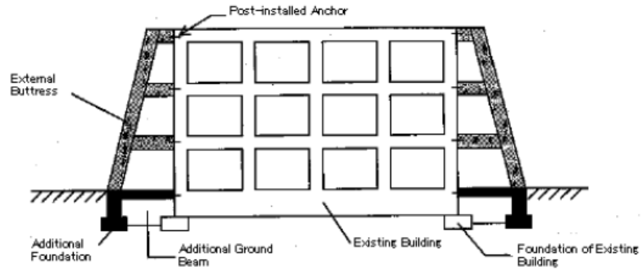


Da Vinch Ginza Building

SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Methods of Retrofitting

7. Constructing External Buttress



SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Base Isolation

Lead Rubber Bearing with Isolator used

Damper used

Thank you very much

Quality Control of Construction Works

Md. Ziaul Hafiz
CNCRP Project

Definition of Quality

- Quality is "meeting or exceeding customer expectations.“
- Quality defined in ISO 8402 as: ‘Totality of characteristics of an entity which bear on its ability to satisfy stated and implied needs.’

Quality Control

- Quality Control is all about ensuring the finished product meets the standards set in the specification and by various controlling bodies.
- **Quality Control**
  - Focuses on the operational techniques and activities used by those involved in the project,
  - Fulfills the requirements for quality (for example, by quality inspections or testing),
  - Identifies ways of eliminating causes of unsatisfactory performance.

Main Objectives of Quality Control in relation to Civil construction Works

- **Scrutiny and interpretation of drawings and specifications.**
- **Liaison with design Engineer**
- **Evaluation of materials and plant**
- **Preparation of construction procedures**
- **Inspection and approval of construction joints, formwork, rebar etc.**

Main Objectives of Quality Control in relation to Civil construction Works

- Supervision of concrete production, placing and curing
- Materials testing
- Preparation of documentation including "as-built" drawings

Phases of Quality Control

1. Quality Control at Design Phase
2. Quality Control at Construction Phase

Quality Control at Design Phase

- Ultimate purpose of any civil construction works is to fulfill requirements of the clients and/or building owner. To realize the purpose, architects, structural engineers, electrical & mechanical engineers and concerned staff have a duty to ensure the design quality. Accordingly, management tools targeting to satisfy the requirement designated in the relative acts and codes shall be used at the time of designing.

Quality Control at Design Phase

- To ensure the quality of the design works, the engineers shall follow some guidelines which include at least-
  - Checklists for design works
  - Corrective action for unsatisfactory design works
  - Record management

Quality Control at Design Phase

❑ Checklists for Design Works:

To realize quality control "Checklist for Design Works" shall be checked by the Structural Designer and Electrical / Mechanical Engineer.

Quality Control at Design Phase

Sample of Design Checklist

CHECKLIST FOR STRUCTURAL DESIGNS

SL. NO.	CHECK ITEM	1ST LEVEL CHK (AE/SDE)				2ND LEVEL CHK (EE)				3RD LEVEL CHK/APPROVAL (SE).			
		YES	NO	UNK/NA	REMARKS	YES	NO	UNK/NA	REMARKS	YES	NO	UNK/NA	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	Are overall plan dimensions indicated in the drawing sheet?												
2	Is topographical survey reviewed?												
3	Is drainage concept described for such as – - sum pit - porch - plain or slanted roof. Etc												
4	Is statement on flood (such as HFL, LFI) considerations provided?												
5	Are the design parameters given? Such as: - SD/USD - fcr/f'c - fy, fs etc.												
6	Is a geotechnical investigation report reviewed?												
7	Is a site plan shown?												
8	Do building lay-outs including column and grid lines provided agree with the corresponding architectural drawings?												
9	Are the plans and sections of existing grades of the land and finished floor elevations provided?												

Quality Control at Design Phase

❑ Checklists for Design Works:

All checklists shall be recorded and stored belonging to the *table below*, so that all concerned persons can easily find out their whereabouts and/or understand the contents.

Quality Control at Design Phase

❑ Checklists for Design Works:

Sample of Checklist record form

Documents title	Storing Place	Storing Period	Responsible person
Structural Drawing	*** Department, PWD	*** years (until the end of year ****)	Mr. ******, Title **** Department
Checked List			

Quality Control at Design Phase

- **Corrective Action for Unsatisfactory Design Works:**
  - **1. Design Engineers have the responsibility for carrying out corrective actions for unsatisfactory design works.**

Quality Control at Design Phase

- **Corrective Action for Unsatisfactory Design Works:**
  - **2. In case of any errors, Design Engineers shall decide the needed action upon consideration of factor of errors.**

Quality Control at Design Phase

Sample of Corrective Action Form

Name of the Project

Date, Month of Error
(DD/MM/YY)
Location
(Name of building, X and Y position, Region)
Work Description
(Name of work with errors)
Detail Description of Error
Factor/Reason of Error
Action to be taken
Date, Month of Correction
Date, Month of Confirmation by Engineer

Date: DD/MM/YY

(Signature)
(Name of the Engineer)

Quality Control at Design Phase

- **Record management:**

The Design Engineers shall record and keep the following documents at the end of both checking & correction-

1. Results of checking

2. Corrective action (if any)

Quality Control at Construction Phase

- Quality control gets its most intensive application in the construction phase. Construction is the area where payoff from quality control is perhaps the greatest.
- With the best designs and specifications, one can still have a poor structure if things go wrong in the field.

Outline of Quality Control at Construction Phase

- Human Control
- Materials Control
- Quality Control by Inspection & Checklist
- Control of Construction Machinery and Equipments
- Control of Construction Methods
- Environmental Control
- Documents Control

Human Control at Construction Phase

As the main activity part of construction process, the overall quality and individual ability of human will determine the results of all quality activities.

Human Control at Construction Phase

The main measures and approach of human control are as follows:

(1) The management objectives and responsibilities of project manager being considered as the center, the organization of project management should be set up reasonably with appropriate management personnel.

Human Control at Construction Phase

The main measures and approach of human control are as follows:

(2) With the strict qualification review of sub-units, the overall quality of sub-units should be controlled, including the technical quality, management quality, service and social reputation.

Human Control at Construction Phase

The main measures and approach of human control are as follows:

(3) The operating workers should be asked certificates, particularly important technical trades, special trades, and aloft work, etc.

Human Control at Construction Phase

The main measures and approach of human control are as follows:

(4) The training, discussions and exchange activities of quality control should be carried out, to strengthen staff's quality consciousness.

Human Control at Construction Phase

The main measures and approach of human control are as follows:

(5) There should be very strict on-site management system and production discipline, and the standard of operation technology and management activities.

Human Control at Construction Phase

The main measures and approach of human control are as follows:

(6) Incentives and communication activities should be promoted to arouse staff's enthusiasm.

Materials Control at Construction Phase

Control of Material quality is one of necessary conditions to ensure construction quality.

Main contents of quality control of materials:

(1) Material Procurement

(2) Material Testing

(3) Material Storage and Usage

Materials Control at Construction Phase

(1) Material procurement

- The procurement should be arranged in advance according to the construction schedule.
- The contractor should purchase materials based on the integrated consideration of engineering characteristics, construction contracts, and the scope of application, construction requirements, the performance and price of materials.

Materials Control at Construction Phase

(2) Material testing:

Frequent testing of construction materials is one of key quality controlling factors of construction. Through a number of tests, the material data obtained is compared with quality standards like BDS, EN, ASTM, ISO, IS etc. to judge the reliability of quality of materials and whether they can be used for engineering construction.

Materials Control at Construction Phase

CNCRP

Testing Apparatus

Materials Control at Construction Phase

CNCRP

Compressive Strength Test of Concrete Cylinder

Materials Control at Construction Phase

CNCRP

(3) Material Storage and Usage:

Proper storage and usage are essential to avoid material deterioration or misuse

Improper storage and usage result in-

a) agglomeration of wet cement

b) corrosion of steel

c) mix of reinforcement with different diameters.

Materials Control at Construction Phase

CNCRP

Storage of Cement

Materials Control at Construction Phase

Storage of Coarse Aggregate

Materials Control at Construction Phase

Storage of Rebar

Quality Control by Inspection at Construction Phase

Inspection means to check the construction works in the middle of construction process.

Objective of Inspection:

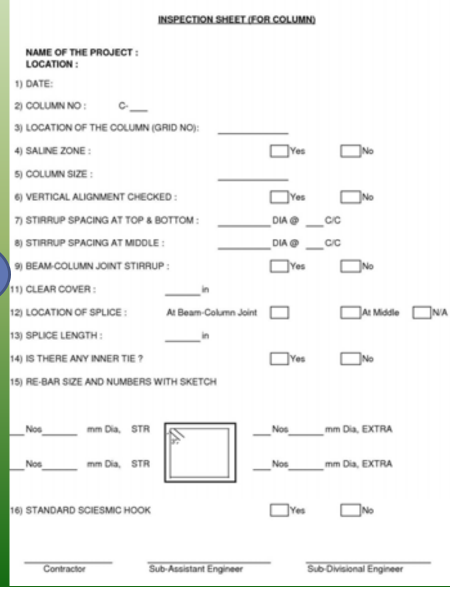
to determine whether the procured materials or completed works meet the standard or not.

Types of Inspection:

1) Total Inspection (100% inspection)
2) Sampling Inspection- all sampling inspection method shall specify frequency.

Quality Control by Inspection at Construction Phase

Sample of Inspection Checklist (for column)



INSPECTION SHEET (FOR COLUMN)

NAME OF THE PROJECT :
LOCATION :

1) DATE:
2) COLUMN NO : C-___
3) LOCATION OF THE COLUMN (GRID NO): ________
4) SALINE ZONE : ☐Yes ☐No
5) COLUMN SIZE : ________
6) VERTICAL ALIGNMENT CHECKED : ☐Yes ☐No
7) STIRRUP SPACING AT TOP & BOTTOM : ________ DIA @ ___ C/C
8) STIRRUP SPACING AT MIDDLE : ________ DIA @ ___ C/C
9) BEAM-COLUMN JOINT STIRRUP : ☐Yes ☐No
11) CLEAR COVER : ______ in
12) LOCATION OF SPLICE : At Beam-Column Joint ☐ ☐At Middle ☐N/A
13) SPLICE LENGTH : ______ in
14) IS THERE ANY INNER TIE ? ☐Yes ☐No
15) RE-BAR SIZE AND NUMBERS WITH SKETCH

___Nos______ mm Dia, STR ___Nos______ mm Dia, EXTRA

___Nos______ mm Dia, STR ___Nos______ mm Dia, EXTRA

16) STANDARD SCIESMIC HOOK ☐Yes ☐No

Contractor Sub-Assistant Engineer Sub-Divisional Engineer

Quality Control by Inspection at Construction Phase

Sample of Construction Checklist

CONSTRUCTION CHECKLIST

NAME OF THE PROJECT :

LOCATION : DATE:

STEP-1: CHECK LIST FOR THE SITE

SL. NO.	CHECK ITEM	Reference	YES	NO	NA	LS Nos	Frequency	1st Level checked SAE	2nd Level Checked SDE	3rd Level Checked E.E	Comments
	Does the proposed site is a private land ?										
	Keeping relevant site hand over papers (having all signatures properly).										
	Approved site plan.										
	Spot level										
	Is there any dispute with nearby land owner and proposed site ?										
	Is there any soil erosion and flood control plan required?										
	If Yes, has it been accepted?										
	Is there any Electric over head line passes over the site ?										
	If Yes, has it been accepted ?										
	Location of adjacent streets and alleys										
	Location / indication of :-										
	temporary labor shed place,										
	water supply arrangement,										
	Material storing place										
	Permanent bench mark										
	Highest flood level										
	Is there any trees and house (to be dismantled) within the site ?										
	If Yes, is there any satisfactory action ?										
	Proper cleaning for foundation layout										

STEP-2: CHECK LIST FOR THE ARCHITECTURAL DRAWING

SL. NO.	CHECK ITEM	Reference	YES	NO	NA	LS Nos	Frequency	1st Level checked SAE	2nd Level Checked SDE	3rd Level Checked E.E	Comments
	Approved architectural drawing.										
	All dimensions are visible and sum of individual dimensions are checked										
	with the total dimension.										
	Is there any part of the architectural drawing that is to be constructed										
	from it's foundation level like cantilever veranda, duct, etc										
	(Not mentioned in structural drawing)										

Quality Control by Inspection at Construction Phase

CHECKING OF RCC WORK

Quality Control by Inspection at Construction Phase

SLAB CHECKING

Control of Construction Machinery and Equipments

Construction machinery and equipments are essential facilities for the modern construction, reflecting the construction power of the enterprise, and having a direct impact on the project progress and quality.

Control of Construction Machinery and Equipments

CNCRP

(1) The contractor should select construction machinery and equipment in accordance with

a) advanced technology,

b) economic rationality,

c) production application,

d) reliable performance and safety,

Control of Construction Machinery and Equipments

CNCRP

(2) The performance parameters should be made sure correctly in accordance with the requirements of construction and quality assurance.

(3) Construction machinery and equipment should be regularly calibrated, so as not to mislead the operator.

Control of Construction Methods

CNCRP

Construction methods are reflected in the concentration of technical solution, process, testing methods and arrangements of construction procedures.

Control of Construction Methods

CNCRP

Main aspects of Construction methods :

(1) Construction program should be constantly refined and deepened with the progress of the project construction.

Control of Construction Methods

Main aspects of Construction methods :

(2) To select the construction program some viable options of major projects should be prepared to choose the best option, presenting

a) main contradictions,

b) advantages and disadvantages,

e) discussion and comparison.

Control of Construction Methods

Main aspects of Construction methods :

(3) When developing programs for the major projects, key and difficult parts of the projects should be fully assessed such as

a) the new structure,

b) new materials,

c) new technology,

d) large-span, large cantilever, the tall structure parts, and so on,

e) the possible construction quality problems and treatment.

Environmental Control

Creating a good environment will play an important role in guaranteeing the quality and safety of construction projects.

Main items of Environmental Control-

a) Control of the natural environment

b) Control of management environment

c) Control of working environment

Environmental Control

(a) Control of the natural environment is to

1) Grasp data and information of hydrology, geology and meteorology of construction site,

2) Know the actual conditions of ground and underground water, affecting construction.

Environmental Control

(2) Control of management environment is to

1) Learn the management relations of all participating construction units.

2) Acquire the coordination, communication and good public relations with the neighboring residents .

Environmental Control

(3) Control of working environment is to

1) Do rational planning and management of construction plan,

2) Arrange the layout of mechanical equipment, materials, components, roads, pipelines, and various large temporary facilities.

3) Take various protective measures.

Documents Control

S/V1 : Documents for commencement work 1.Commencement Order 2.Hand over letter to Contractor, etc. : Construction Management Manual - Prepared by Consultant - : Work Plan by Contractor	S/V5 : Weekly Report by Contractor 1. Minutes of Meeting 2. Weekly Progress Schedule 3. Quality Control Report, if necessary 4. Performance Control Report, if necessary 5. Safety Control Report, if necessary
S/V2 : Communication Letter 1.Question and Answer letter with Client 2.Question and Answer letter with Contractor	S/V6 : Monthly Report 1. Submit to Client , if necessary
S/V3 : Quality Control -Approved Document- 1. Materials : a. Certificate of Each Materials, Mill Sheet, Frog Mark of Brand, etc. by Contractor b. Testing Report Include Picture by PWD 2. Shop Drawings by Contractor, if possible	S/V7 : Document of Contract and Payment 1. Request Letter from Concerned Authority 2. Contract of Contractor 3. Payment Record
S/V4 : Performance Control - Inspection Report by Contractor, approved by PWD - 1. Inspection Record include Pictures – Architectural work - 2. Inspection Record include Pictures – Mechanical work - 3. Inspection Record include Pictures – Electrical work - 4. Inspection Record include Pictures – Plumbing work -	S/V8 : Completion Document 1. As- built Drawing by Contractor 2. Final Inspection Report by Consultant 3. Completion Report by Consultant

Documents Control

1 Documents for commencement Work

- Work Order
- Hand Over letter to contractor
- Management Guideline by consultant
- Work plan by contractor etc.

2 Communication letters

- Question and answer letter with client
- Question and answer letter with contractor.

Documents Control

3 Quality Control- Material approved documents

- Certificates of each materials, Mill sheet, Frog mark of brand etc. by contractor.
- Testing Reports including pictures.
- Shop drawing by contractor, if necessary.

4 Performance Control

- Inspection Report by contractor approved by the consultant/ authority
- Inspection Record include Pictures

Documents Control

5 Weekly Reports by Contractor

- Minutes of Meeting
- Weekly Progress Schedule
- Quality Control Report, if necessary
- Performance Control Report, if necessary
- Safety Control Report, if necessary

6 Monthly Report

- Monthly report submit to client.

Documents Control

7 Documents of Contract and Payment

- Request Letter from concerned authority
- Contract of contractor
- Payment Records

8 Completion Documents

- As-built drawing by Contractor
- Final Inspection Report by Consultant
- Completion Report by Consultant

Thank you very much

Management of Retrofitting Construction

Md. Mafizur Rahman
And
Md. Sohel Rahman
CNCRP Project

Retrofitting Work

What are special considerations!

Restraint work condition

- Small scale construction/ Part by part
- Limited access due to occupier
- Dust and Noise control
- Limited Time etc

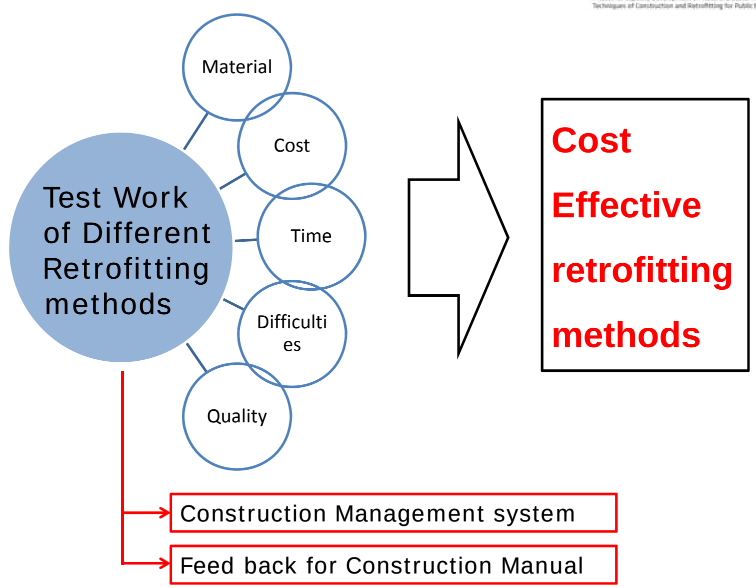
Retrofitting Work

What are special considerations!

Knowing New Techniques & Materials

- Post installed anchor work.
- Non shrink Grout Work.
- Use of construction chemicals to improve workability, strength etc.

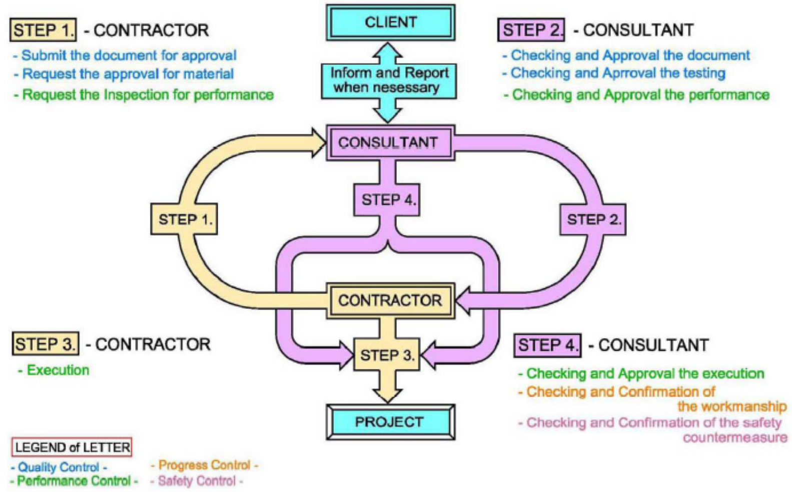
Retrofitting Test Work



Outline of Construction Management

- Documents Control (File Management)
- Quality Control- Material
- Quality Control- Performance
- Progress Control
- Safety Control

Flow Chart : Construction Stage



Documents Control

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Documents Control

1 Documents for commencement Work

- Work Order
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2 Communication letters

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Documents Control

3 Quality Control- Material approved documents

- Certificates of each materials, Mill sheet, Frog mark of brand etc. by contractor.
- Testing Reports including pictures.
- Shop drawing by contractor, if necessary.

4 Performance Control

- Inspection Report by contractor approved by the consultant/ authority
- Inspection Record include Pictures

Documents Control

5 Weekly Reports by Contractor

- Minutes of Meeting
- Weekly Progress Schedule
- Quality Control Report, if necessary
- Performance Control Report, if necessary
- Safety Control Report, if necessary

6 Monthly Report

- Monthly report submit to client.

Documents Control

7 Documents of Contract and Payment

- Request Letter from concerned authority
- Contract of contractor
- Payment Records

8 Completion Documents

- As-built drawing by Contractor
- Final Inspection Report by Consultant
- Completion Report by Consultant

Quality Control- Materials

- A Quality Control Program must be set by the consultant as per design and specification, such as Quality Control Table, Performance Control Table etc.
- Confirmation of quality of materials as per the quality control program

Quality Control- Materials

QUALITY CONTROL TABLE

WORK	CHECK ITEM	CHECK METHOD	STANDARD	FREQUENCY OF CHECKING	TREATMENT OF RESULTS	REMARKS
I CONCRETE						
A Material						
1 Cement	1) Classification of Cement 2) Match to the Standard	Printing of the cement bag Printing of the cement bag	i) OPC: BDS-EN-197-1:1993 CEM-I, 42.5 N ii) PCC: BDS EN-197-1:1993 CEM-II/A,	Occasionally at Plant's Store Prior to Mixing Design		
2 Admixture	1) Type, Suitability	Manufacturer's Specification	BDS EN-934-1 / Manufacturers Standard	Prior to Mixing Design	Submit Manufacturer's Specification	
3 Water	1) Quality for concrete mixing	JIS A5308 or equiv. Confirmation by Tested Results	Ionized Chloride is not more than 200ppm BDS: Potable Water	Prior to Mixing Design	Submit Test Report	Tested by Third Party Laboratory
4 s	1) Grading Range	JIS A 1102 or equiv. Confirmation by Tested Results	JASS 5 Table 4.3 BDS 243 (1963)	Once when source or kind of aggregates are changed	Submit as "Test Report of Aggregates"	Witnessed by the Consultant If Absorption, Grading Range, Unit Weight and Specific Gravity exceed the specified range, mixing design shall be adjusted accordingly.
	2) Unit Weight of Aggregate	JIS A 1104 or equiv. Confirmation by Tested Results	1.25kg/L			
	3) Specific Gravity & of Fine Aggregate	JIS A 1109 or equiv. Confirmation by Tested Results				
	4) Specific Gravity & of Coarse Aggregate	JIS A 1110 or equiv. Confirmation by Tested Results				
	5) Muddy Substance after Washing Aggregates	JIS A 1103 or equiv.				
	6) Alkali-Aggregate Reaction	JIS A 5308 (Appendices 7, 8) or equiv.	Judged as "non-harmful" to concrete			If judged as harmful, counter measure to suppress Alkali-Aggregate Reaction shall
	7) Amount of Chlorides	Supervision Guidance for Construction of Building by the Minister's Secretariat, Ministry of Land, Infrastructure and	JIS A 5308 or equiv. Total Amount of Chlorides: ≤0.3kg/m3			If exceed the allowable value, adequate counter-measure shall be taken.
B Mixing	1) Trial Mixing	Mixing Design Sheet	TS: Chapter 5.4.1 (Slump, Temperature, Strength)	Once at the time of Commencement with each strength and whenever mixing design	Submit Test Report	Witnessed by the Consultant

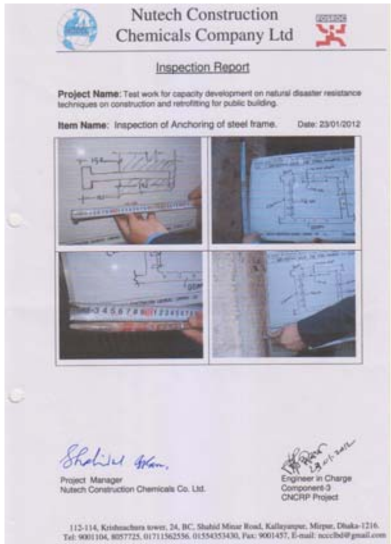
Quality Control- Materials

QUALITY CONTROL TABLE			
WORK	CHECK ITEM	CHECK METHOD	STANDARD
CONCRETE			
A Material			
1 Cement	1) Classification of Cement	Printing of the cement bag	i) OPC: BDS-EN-197-1:1993 CEM-I, 52.5 N
	2) Match to the Standard	Printing of the cement bag	
2 Admixture	1) Type, Suitability	Manufacturer's Specification	BDS EN-934-1 / Manufacturers Standard

Quality Control- Materials

	QUALITY CONTROL TABLE	(Continued)	
	FREQUENCY OF CHECKING	CONTRACTTOR'S RESPONSIBILITY	REMARKS
1 Cement	Occasionally at Plant's Store prior to mixing design	Approval to be taken before mixing	
2 Admixture	Prior to Mixing Design	Manufacturer's specification and previous test record to be submitted before mixing for approval	

Quality Control- Materials


Nutech Construction Chemicals Company Ltd

Inspection Report

Project Name: Test work for capacity development on natural disaster resistance techniques on construction and retrofitting for public building.

Item Name: Inspection of Anchoring of steel frame. Date: 23/01/2012

Project Manager
Nutech Construction Chemicals Co. Ltd.

Engineer in Charge
Component-3
CNCRP Project

112-114, Krishnachura tower, 24, BC, Shahid Minar Road, Kallayanpur, Mirpur, Dhaka-1216.
Tel: 9001104, 8057725, 01711562536, 01554353430, Fax: 9001457, E-mail: ncccltd@gmail.com

Quality Control- Performance

o Confirmation of quality of performance as per the performance control program

Quality Control- Performance

CHECK LIST FOR THE PERFORMED WORK

WORK	MEASUREMENT ITEM	METHOD	FREQUENCY OF MEASUREMENT	UNIT mm	TREATMENT OF RESULTS	ALLOWABLE TOLERANCE	REMARKS
A Earthwork							
1 Excavation	Bottom Level	1	• 4 corner and 1 center for square trench • every 5m at center of trench • every 5m in length and breadth for overall excavation	10	Survey Record	±30 mm	Photo record shall be attached
2 Backfilling	Top Level	1	• every 5m in length and breadth	10	Survey Record	±30 mm	Photo record shall be attached
B Foundation Work							
1 Gravel /Crushed Stone	Top Level	1	• 4 corner and 1 center for square trench • every 5m at center of trench • every 5m in length and breadth for overall excavation	10	Survey Record	±30 mm	Photo record shall be attached
	Width	2	Every 5m	10	Survey Record	±50 mm	Photo record shall be attached
C Concrete Work							
1 Footing	Top Level	1	Random	1	Survey Record	±20 mm	Photo record shall be attached
Underground Beam	Cross Section Dimensions	2	Random	1	Survey Record	+50 ~ -10 mm	Photo record shall be attached
2 Slab on Grade	Top Level	1	Random	1	Survey Record	±10 mm	Photo record shall be attached
3 Column	Cross Section Dimensions	2	Random	1	Survey Record	+20 ~ -5 mm	Photo record shall be attached
	Deviation from Plumb Line	2	Random	1	Survey Record	±20 mm	Photo record shall be attached
4 Girder, Beam	Cross Section Dimensions	2	Random	1	Survey Record	+20 ~ -5 mm	Photo record shall be attached
	Bottom Level	1	Random	1	Survey Record	±20 mm	Photo record shall be attached
5 Slab	Top Level	1	Random	1	Survey Record	±10 mm	Photo record shall be attached
	Bottom Level	1	Random	1	Survey Record	±20 mm	Photo record shall be attached

Quality Control- Performance

PERFORMANCE CONTROL TABLE

WORK	MEASUREMENT ITEM	METHOD	FREQUENCY OF MEASUREMENT
D Structural Work			
1 Reinforcement Work	Diameter, number and space	As per Design Drawings	When completed assembling re-bars.
	Length and location of splice joints, Anchor length	As per Design Drawings	When completed assembling re-bars.
	Concrete coverage, Additional bar	As per Design Drawings and Specifications	When completed assembling re-bars.

Quality Control- Performance

PERFORMANCE CONTROL TABLE (Contd.)

	UNIT mm	TREATMENT	ALLOWABLE	REMARKS
		OF RESULTS	TOLERANCE	
1 Reinforcement Work		Inspection Record		Photo record shall be attached
				Photo record shall be attached
				Photo record shall be attached

Quality Control- Performance


Inspection Report

Project Name: Test work for capacity development on natural disaster resistance techniques on construction and retrofitting for public building.

Item Name: Inspection of Carbon fiber system. Date: 17/01/2012

Apply of primer (Nitowrap 30)
Measure & cutting of Nitowrap EP (CF)
Mixing of Nitowrap 410, Saturant
Appling of Nitowrap EP(CF) with Nitowrap 410, Saturant

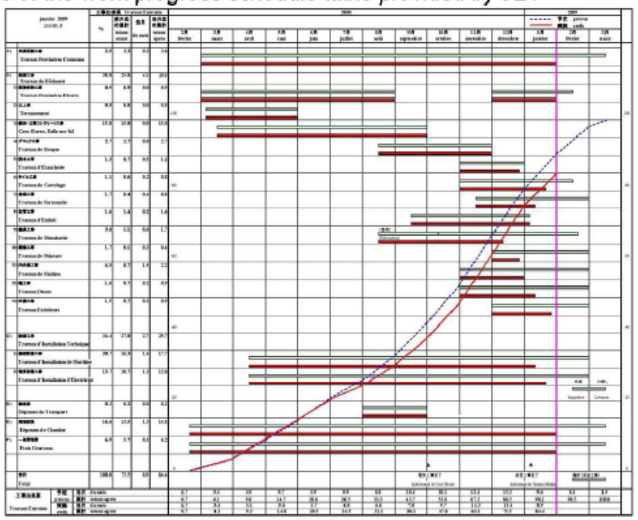
- Strength test of anchor by hammering
- Site: Chiba Prefectural Sakura Higashi Senior High School

Progress Control

- o Request the contractor to submit the work progress chart/ graph (Time schedule).
- o Confirm the work progress at the weekly meeting, on schedule or not.
- o When find out the problem of progress, discuss with the contractor how to solve.

Progress Control

Example of the work progress schedule table provided by JET



SOURCE: Class note of Mr. Takeshi TAKESHITA for CNCRP Project

Safety Control

- o Confirm the safety measures taken for workers and local residents.
- o Make sure crisis management is perfect, and confirm the emergency response

Safety Control

- Green Mark
- Site: IDEC Corporation new HQ

- Worker with Safety Harness
- Site: IDEC Corporation new HQ

Thank you very much

4th Domestic Training

Venue: PWD Seminar Room

and Site Visit (Tejgaon Fire Station)

Duration: Nov 12, 13 and 15 2014

Short Training Course on

"Techniques of Retrofit Construction for R.C. Buildings and Quality Control for Retrofitting Work"

Under "RMG Sector Safe Working Environment Program"

Attendence Sheet

Date: ~~13~~ 15.11.2014

SL. NO	Engineers Name	Name of company / Applicant	Designation	Contract No	E-mail	Signature	Remarks
1	Md.Rahmat-E-Rabbi	d.zign Scape Consultants Ltd.	Construction Engineer	(+88) 01826474747	rabbi@dzignscape.net	15/11/14	
2	~~Nur~~ Noor Mohammad	d.zign Scape Consultants Ltd.	Assistant Engineer	(+88) 01822441999	noorrangpur@gmail.com		correction
3	Md. Anowarul Mahmud	Associated Builders Corporation Ltd	Project Manager	(+88) 01819246789	biplob_90@yahoo.com	15/11/14	
4	Md. Abu Shoeb	Associated Builders Corporation Ltd	Head of Project Evaluation	(+88) 01817537482	abcltd72@gmail.com	15/11/2014	
5	Md.Nizamul Haque	M/S Troyee Enterprise	Project Manager	(+88) 01823061374 (+88) 01711817029	nizamulhaque.milon@gmail.com		
6	Md.Jakir Hossain	M/S Troyee Enterprise	Site Engineer	(+88) 01733019677	jakir9602@gmail.com	Hossain	
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8	Md.Saiful Islam	The Cementation	Management Trainee	(+88) 01712717522	shohag39@~~yahoo.com~~ gmail.com	Shohag	
9	Md.Abdul Ali	Padma Associates & Engineers Ltd	Site Engineer	(+88) 01922338975		Ali 15/11/14	
10	Md. Abdul Kader Mia	Padma Associates & Engineers Ltd	Senior Site engineer	(+88) 01913621120		15.11.14	
11	Md.S~~h~~aidur Rahman Chowdhury	Auspicious	Senior Structural engineer	(+88) 01911312911	md.saidur@auspiciousbd.com		Name correction
12	Md.Anoyar Bin Rashid	Auspicious	Site Engineer	(+88) 01723742472	anoyar@auspiciusbd.com		

13	Md.Saiful Islam	Heritage housing Ltd.	Structural engineer	(+88) 01811421041	saiful602010@gmail.com		
14	Md.Abu Hasan	Heritage housing Ltd.	~~Project Engineer~~ Managing Director	~~(+88) 0181628264~~ 01816288264	heritage64@gmail.com haritage64@gmail.com		cel No: 8 Email No change
15	Sk.Kamal Hossain	Engineering & Supports Services	Project Manager	(+88) 01714046086	kamal_engg2006@yahoo.com		
16	Animesh Chandra Biswas	Engineering & Supports Services	Project Co-ordinator	(+88) 01713435005	animesh.kus@gmail.com		
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18	Shaikh raseduzzaman	Shunirman Associates & Builders Ltd.	Assistant Engineer	(+88) 01718017200	rased1982@gmail.com		
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20	Amit Kumar Bhadra	National Development Engineers Ltd	Office Engineer	(+88) 01712110921	samit95079@gmail.com	Amit	
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22	~~Puulak Kumar Basu~~	United Engineering Consortium	COO				
23	Md. Sharegul alam	Greenyard construction Ltd.	Managing Director	(+88) 01989155361	greenyardltd@gmail.com		
24	Md.Forjul Islam	Greenyard construction Ltd.	Project Manager	(+88) 01918042104	greenyardltd@gmail.com		
25	Md.Shamim Haider Khan	Construction Aid & Logistics Ltd	Junior Manager	(+88) 01911771734	shamim_buet05@yah00.com		
26	Md.Nasir Uddin	Construction Aid & Logistics Ltd	Purchase Engineer (Sells & Marketing)	(+88) 01755607063	nasircall@gmail.com		
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30	Mohammad Wali ~~Ulah~~ Ullah	Eclectic	Project Engineer	(+88) 01818485023	wali99ullah@gmail.com ~~wali99_ullah@gmail.com~~		Print again Name Correction
31	Md.Azizul Islam Khan	City Steel Building	General Manager	01721596972	mittumaimun13@gmail.com		
32	Saidur Rahman	City Steel Building	Project co-ordinator	01720319624	citysteelbuilders@yahoo.com	Saidur Rahman	
33	Hilton Das	Retrotech Associates	Advisor	(+88) 01713336282	arkoconsults@gmail.com		
34	Saikat Roy	Retrotech Associates	Project Engineer	(+88) 01979225697	saikat.k.roy@gmail.com		
35	Engr.Aminul Haque Bhuiyam	The Civil Engineers Ltd	Project Engineer	(+88) 01714103079	eaminul@gmail.com		
36	Md.Shahidul Islam	The Civil Engineers Ltd	Seinior Engineer	(+88) 01711638890			
37	Ajmail Hossain	Nutech Construction Chemical Co.Ltd	Project Manager	(+88) 01556550063			
38	Durjoy biswas	Nutech Construction Chemical Co.Ltd	Assistant engineer	(+88) 01713238310	durjoybiswas@gmail.com		
39	Md.Shahariar Islam	Hasan & Sons Ltd	Project Engineer	(+88) 01719571959	shahariar.ecn10@gmail.com		
40	Md.Mostafizur Rahman	Hasan & Sons Ltd	Seinior Project Engineer	(+88) 01718180612	shohel.sahi@gmail.com		
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43	Ashique Ahmed	Star Delta Engineers Ltd	Engineer (Civil)	(+88) 01674990900	ashikbsce@gmail.com	Ashik Ahmed	
44	Md.Abdul kader Sarker	Star Delta Engineers Ltd	Project Engineer	~~(+88) 01722431917~~ 01711027995		15/11/14	
45	Md. Mehedy Hassan	BKMEA	Engineer (Civil)	~~(+88) 01711027996~~ 01722431917	engr.mehedy@gmail.com	Mehedy	
46	Md. Humayun kabir	BKMEA	Engineer (Civil)	(+88) 01920565191	humayun_ce@yahoo.com	Humayun	
47	MD. Shamim Ahammed	Florida Properties Ltd / United Engineer Consult.	Site Engg.	01926095917			
48							

Course Title: **Short Training Course on "Techniques of Retrofit Construction for R.C. Buildings and Quality Control for Retrofitting Work"**

Organizer: CNCRP

1. Course Duration: 3 half-working days

   Lectures 2 days   2:30pm-5:30pm
   Site visit 1 day

2. Participants: Civil Engineers from enlisted Contractors for "RMG Sector Safe Working Environment Program
   (Total 48 participant)
3. Venue: Room No. 725, Purto Bhaban, Segunbagicha, Dhaka-1000

Course content:

1. Introduction to Seismic Retrofitting
2. Outline of Seismic Assessment and Retrofit Design
3. Retrofit Works and Quality Control
4. Management of Retrofit works
5. Site visit

Course content:

Date	**Lecture-1 (2:30-3:20)**	**Lecture-2 (3:20-4:10)**	**Break**	**Lecture-3 (4:30- 5:20)**
12/11/14	Intro. to Seismic Retrofitting :PMT	Retrofitting Methods :WT-3	4:10 – 4:30	Retrofitting Methods (continued) :WT-3
13/11/14	Outline of Seismic Assessment & Retrofit Design :WT2	Quality Control of Retrofit Works :WT-4		Management of Retrofit Construction :WT-3, PMT

15/11/14	**Site Visit (10:00-1:00)** Test site at PWD premise and Retrofit construction of Tejgaon Fire Station	Certificate Awarding

Contact Person:

1. Md. Sohel Rahman
   Executive Engineer
   PWD Design Division-4 and
   Team Leader, WT-3, CNCRP
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   e-mail: sohelr86@yahoo.com

2. Md.Hafizur Rahman.
   Project Engineer, CNCRP
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   e-mail: topu_014@yahoo.com

Introduction to Seismic Retrofitting

Md. Abdul Malek Sikder
Additional Chief Engineer
Planning and Special Project
Public Works Department
&
Project Director, CNCRP

CNCRP

Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

Project Purpose

To develop the capacity of PWD for construction and retrofitting works of the public buildings against natural disasters, such as earthquake, cyclone, flood and high tide.

Concept of the Project

- This project is Technical Cooperation / Support Project for PWD.
- PWD is the Main Actor.
- JICA Expert Team (JET) is the Supporter.
- PWD and JET collaborate to attain the Project Purpose.

What is Retrofitting

- Repair
- Renovation
- Strengthening

Retrofitting is the modification of existing structures to make them more resistant

Seismic retrofitting is the modification of existing structures to make them more resistant to seismic activity, ground motion, or soil failure due to earthquake

Overview

In Standing Orders on Disaster (SOD) 2010, PWD have been entrusted with the following works, in addition to its normal duties, for risk reduction:

- Ensure proper execution of BNBC
- Inclusion of disaster risk in the policies, programmes and guidelines of all the development works of PWD.
- Preparation of manual for seismic capacity evaluation and earthquake resistant design

Overview

- Preparation and periodically updating list of vulnerable structures
- Disseminate technical information related to earthquake and tsunami to engineers
- Support the retrofitting works

Why Retrofitting is Requred

- Buildings constructed before 1993 not following Building Code
- Buildings constructed after 1993 not considering earthquake load
- Buildings designed following Building Code but not implemented properly during construction.
- Buildings designed and constructed following Building Code but retrofitting is required due to changes in the requirements of updated Building Code

Concept of Seismic Retrofitting

Improvement of

- Strength
- Ductility

Strength

Strength of a structural member is its ability to withstand an applied stress without failure

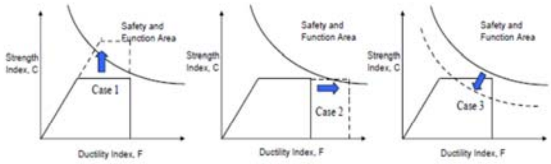
Ductility

Ductility is the ability of structure or its components to provide resistance in the inelastic domain of response.

It includes the ability to sustain large deformations and a capacity to absorb energy by hysteretic behavior.

Concept of Seismic Retrofitting - Combination of strength and ductility

1 Increasing strength (Case 1 of following figure)
2 Increasing ductility (Case 2 of following figure)
3 Improvement of configuration (Case 3 of following figure)
4 Reduction of seismic load (Case 3 of following figure)



[Source: "Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001", The Japan Building Disaster Prevention Association (English version)]

Seismic Index of Structure (Is),

$I_S = E_0 \times S_D \times T$

E_0 - Basic seismic index of structure = **C x F**

C - Strength index = Shear strength/Axial force

F - Ductility index = Same concept of R in BNBC

S_D - Irregularity index

T - Time index

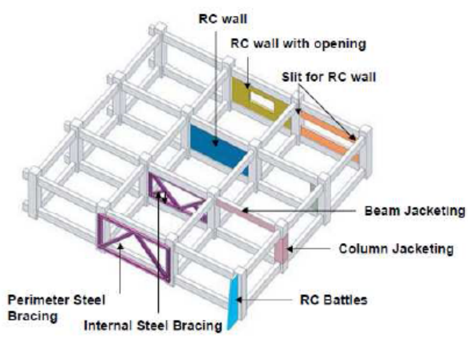
Check : 1) $I_S \geq I_{So}$ 2) **Minimum strength of structure**

I_{So} - Seismic demand index of structure which is a preset value

I_S is checked

- Each story
- Each principal horizontal direction of a building.

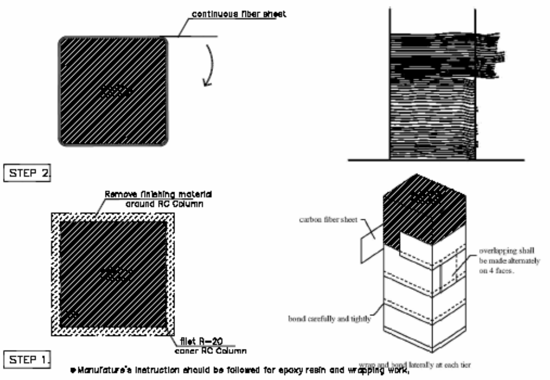
Methods of Retrofitting



RC column jacketing in Japan

RC column jacketing (Test work, CNCRP, Bangladesh)

Retrofitting with Carbon fiber sheet wrapping



Carbon fiber sheet wrapping (Test Work, Bangladesh)

- Carbon fiber sheet wrapping around column

RC wing wall (Test work, CNCRP, Bangladesh)

Improvement type

- Strength

RC shear wall (Test work, CNCRP, Bangladesh)

Improvement type

- Strength
- Structural Balance

Retrofitting with Steel Frame in Japan

Steel framed bracing

a)Junior High School Building (School Colored)

b) Junior High School Building (Designed by Students)

c)University Building (with Exterior Panels)

d)Elementary School Building (with Balconies)

Steel framed bracing (Test work, CNCRP, Bangladesh)

Improvement type

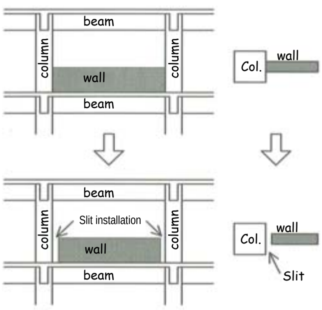
- Strength

Retrofitting with Steel Jacketing in Japan

Slit on brick wall(Test work, CNCRP, Bangladesh)

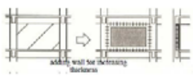
Improvement type

- Ductility
- Structural balance



Methods of Retrofitting

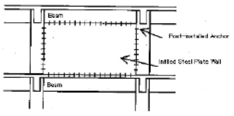
3. Increasing Thickness of Existing Shear Wall



Da Vinch Ginza Building

Methods of Retrofitting

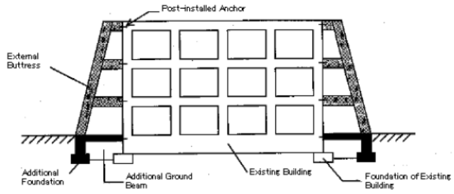
4. Infilling Steel Plate Wall into Open Frame



Under Construction

Methods of Retrofitting

7. Constructing External Buttress

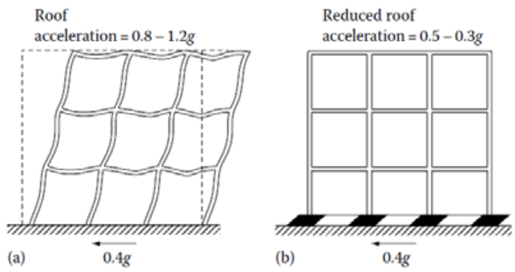


Base Isolation

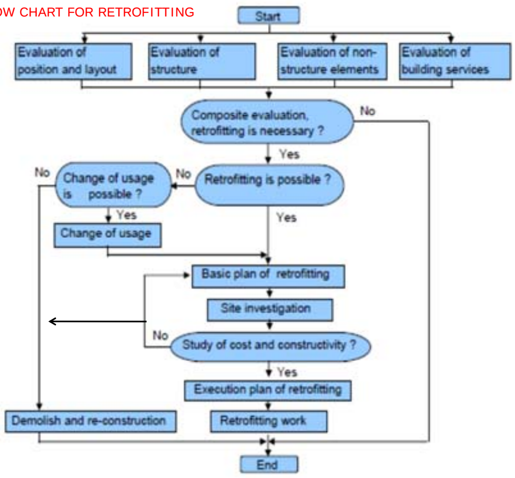
Lead Rubber Bearing with Isolator used

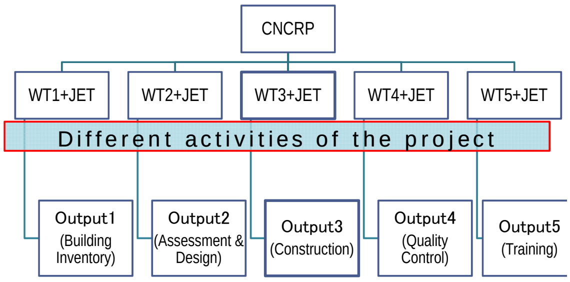
Damper used

Base Isolation



FLOW CHART FOR RETROFITTING





Expected outputs after completion of CNCRP Project

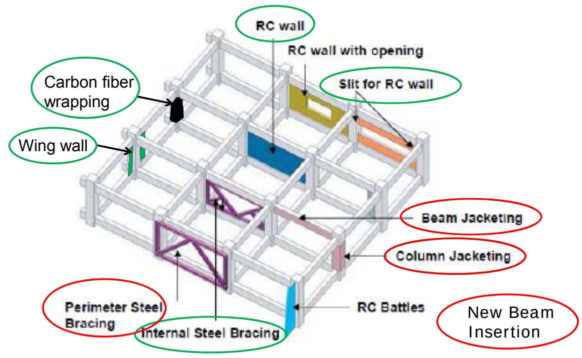
- PWD will have the capacity to do seismic assessment, retrofitting design and construction of existing buildings.
- PWD Training Academy will arrange the training program on 'Retrofitting of Buildings' regularly.
- 6 different manuals on seismic design of new buildings, seismic assessment, retrofitting design, non seismic hazards, construction and quality system will be available.

Thank you

Retrofitting Works

Md. Sohel Rahman
Executive Engineer
PWD Design Division-4
and
Team Leader, Working Team-3
CNCRP Project

Methods of Retrofitting



Retrofitting with Column Jacketing

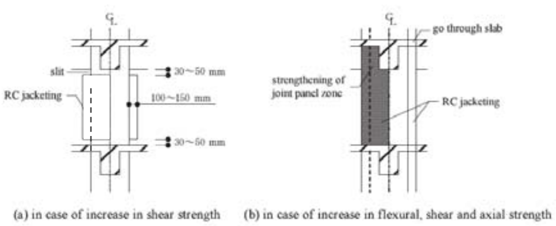
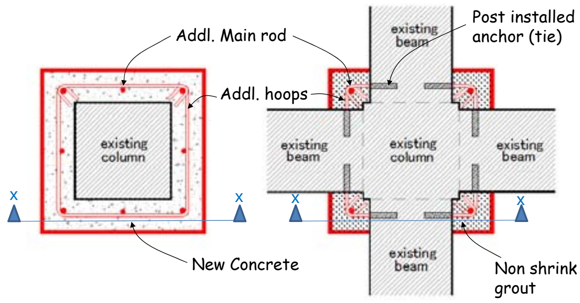


Figure TN.16 Column strengthening with RC jacketing

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

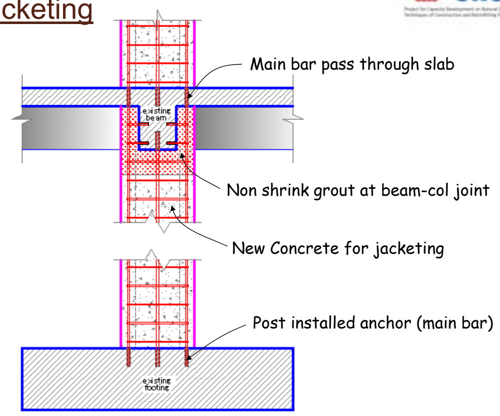
Retrofitting with Column Jacketing



Section of column at mid span

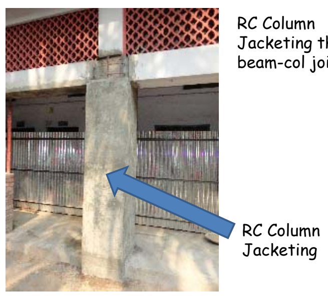
Section of column at beam-col joint

Retrofitting with Column Jacketing

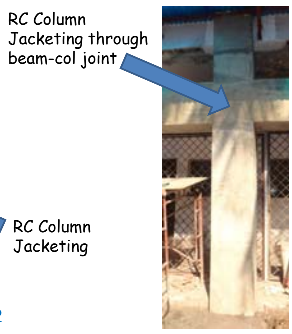


Section of column jacketing

Retrofitting with Column Jacketing



Test Work of CNCRP in 2012



Test Work of CNCRP in 2013

Pics from Pilot Work (Column Jacketing)


Column Jacketing at Gr. Floor


Checking of Reinforcement

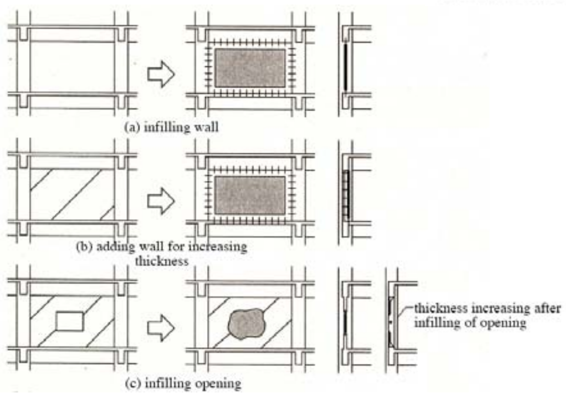

Chemical Applying on old Column Surface


Column Jacketing Work at site

Retrofitting with Column Jacketing

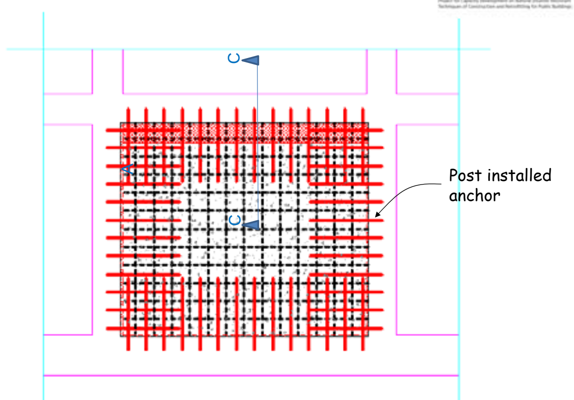
- Improve shear strength and ductility of existing building.
- To improve bonding between old and new concrete provide shear key and/ or epoxy coating over existing column.

Retrofitting with Shear Wall



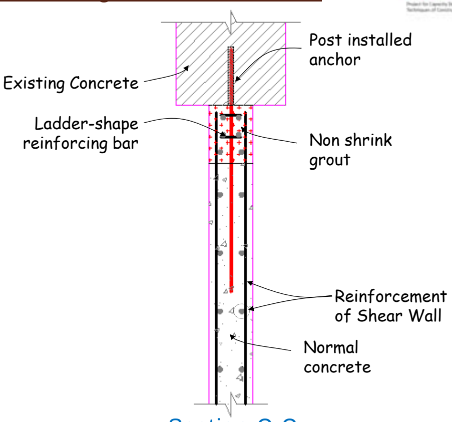
SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Retrofitting with Shear Wall



Details of RC Shear wall

Retrofitting with Shear Wall



Section C-C

Retrofitting with Shear Wall

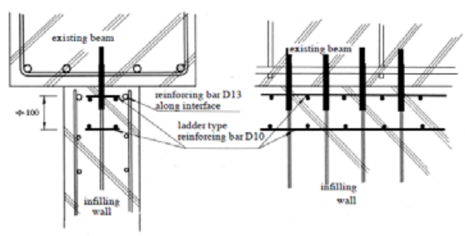


Figure TN.12 Strengthening against splitting with ladder type reinforcing bars
(quoted from the figure on page 98 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Retrofitting with Shear Wall

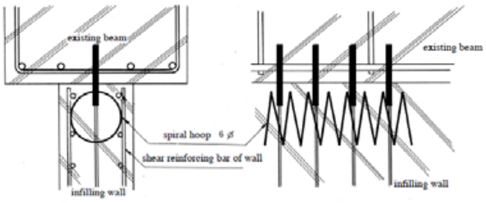


Figure TN.11 Strengthening against splitting with spiral reinforcing bars
(quoted from the figure on page 98 in the commentary of 3.1.4 of the Guidelines of 2001 Japanese version)

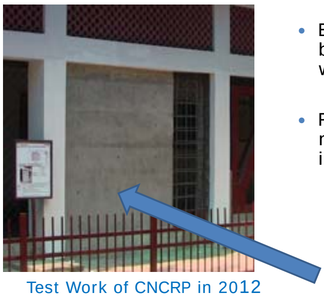
SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001.
Published by- The Japan Building Disaster Prevention Association

Pics from Pilot Work (RC Shear Wall)

RCC Shear Wall at Ground Floor

Retrofitting with Shear Wall

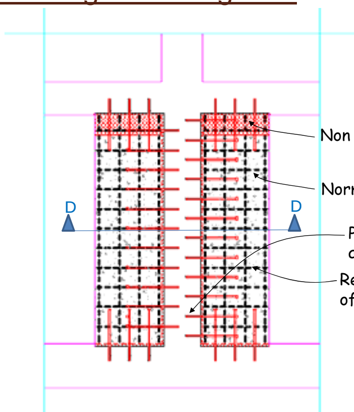
- Improve shear strength and structural balance.
- Existing brick wall may be replaced by RC shear wall.
- Reduces ventilation and natural light if it is used in open frame.



Test Work of CNCRP in 2012

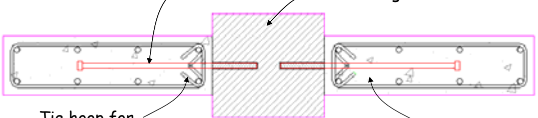
Providing RC Shear Wall in a open frame

Retrofitting with Wing Wall



Details of RC Wing wall

Retrofitting with Wing Wall



Section D-D

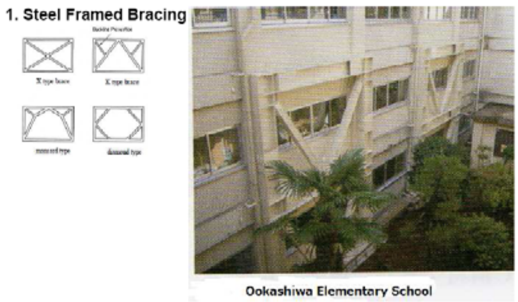
Retrofitting with Wing Wall

- Improve shear strength and ductility.
- failure mechanism from column yielding to beam yielding.
- Not suitable for short span beam.

RC Wing Wall Provided at an existing column

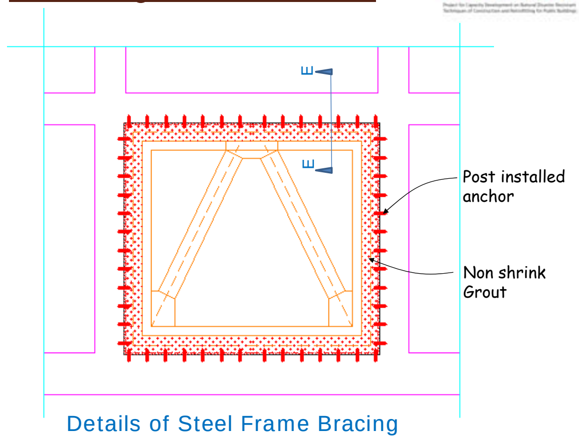
Test Work of CNCRP in 2012

Retrofitting with Steel Frame



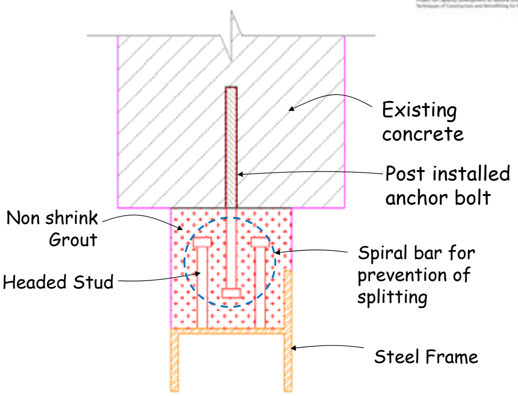
SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Retrofitting with Steel Frame



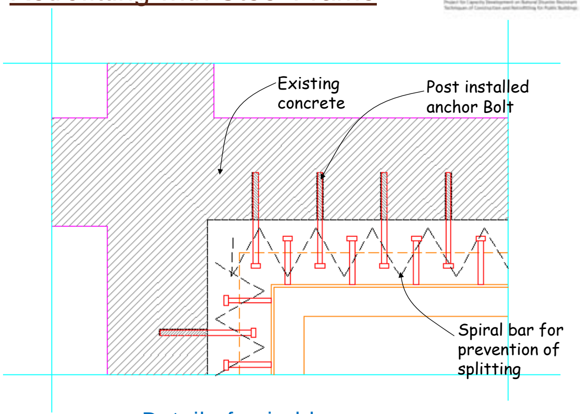
Details of Steel Frame Bracing

Retrofitting with Steel Frame



Section E-E

Retrofitting with Steel Frame



Detail of spiral bar

Retrofitting with Steel Frame

Internal Steel Frame Bracing

Test Work of CNCRP in 2012

External Steel Frame Bracing

Test Work of CNCRP in 2013

Connection Details, Test Work 2012

Steel framed bracing

a)Junior High School Building (School Colored)

b) Junior High School Building (Designed by Students)

c)University Building (with Exterior Panels)

d)Elementary School Building (with Balconies)

SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Pics from Pilot Work (Steel Frame Bracing)

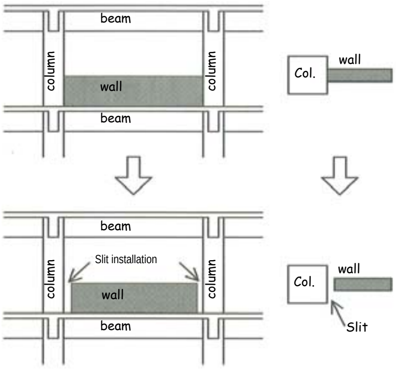
Steel Frame Bracing at Ground Floor

Retrofitting with Steel Frame

Test Work of CNCRP in 2012

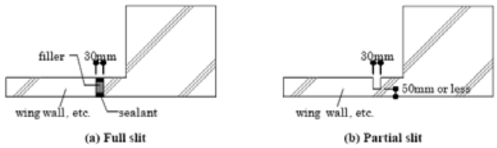
- Improve shear strength.
- soft storey/ weak storey problem may be resolved.
- Lighting and ventilation is not much disturbed, so suitable for outer frames.

Retrofitting with Structural Slit



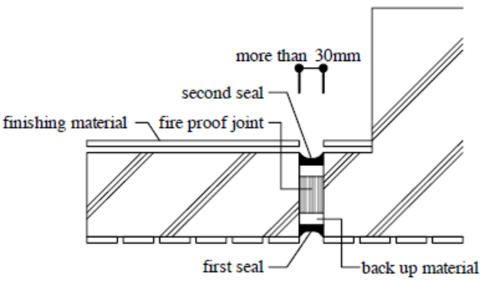
SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Retrofitting with Structural Slit



SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Retrofitting with Structural Slit



Detail of Seismic Slit

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

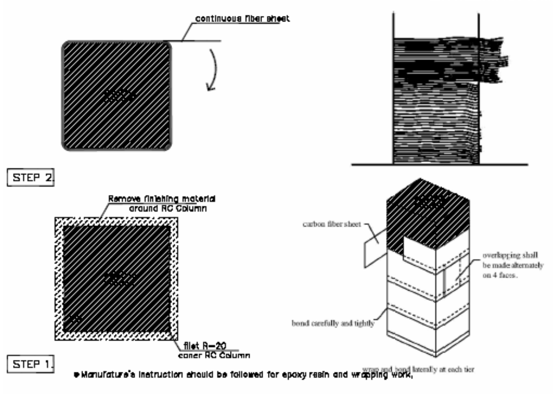
Retrofitting with Structural Slit

Seismic Slit is provided at a brick wall

Test Work of CNCRP in 2012

- To improve structural balance and ductility of existing building
- Short column failure may be avoided.

Carbon fiber sheet wrapping



Carbon fiber sheet wrapping

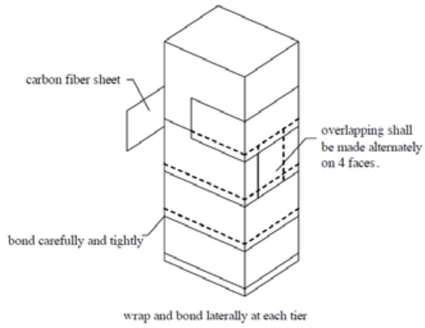


Figure TN.28 Strengthening with carbon fiber sheet wrapping

SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Carbon fiber sheet wrapping

Test Work of CNCRP in 2012

Test Work of CNCRP in 2012

Carbon fiber sheet wrapping

Figure 4.9-1 Flow of standard construction procedure

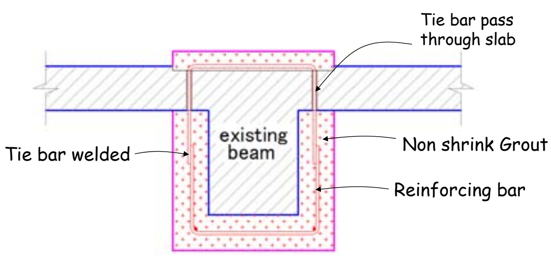
SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Carbon fiber sheet wrapping

Test Work of CNCRP in 2012

- Improve ductility.
- Skilled worker is mandatory to ensure quality construction.
- This method may be applicable on the frames with inadequate seismic hoops or tie

Retrofitting with Beam Jacketing



Typical Detail of Beam Jacketing

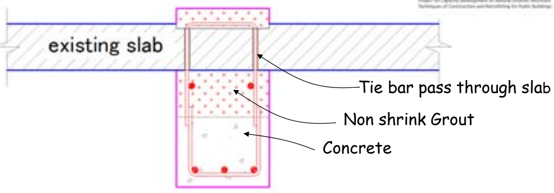
Retrofitting with Beam Jacketing

RC Beam Jacketing

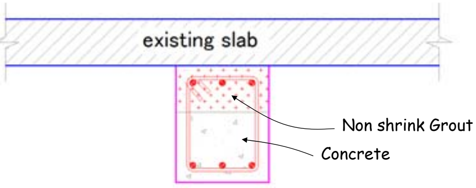
Test Work of CNCRP in 2013

- Improve ductility.

Retrofitting with Beam Insertion



Typical Detail of Beam Insertion (Option-1)



Typical Detail of Beam Insertion (Option-2)

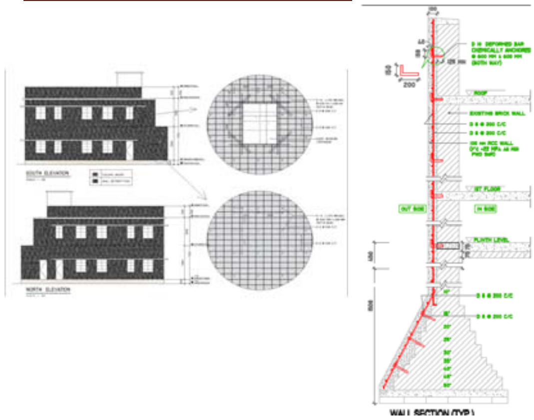
Retrofitting with Beam Insertion

Beam is inserted below existing slab

Test Work of CNCRP in 2013

- Improve ductility and stiffness of the existing building.

Retrofitting with RC Lamination of Brick Wall



Retrofitting with RC Lamination of Brick Wall

RC Lamination at 10" Load Bearing Brick Wall

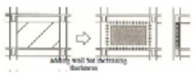
Reinforcement for RC Lamination

Pilot Work of CNCRP in 2014

Pilot Work of CNCRP in 2014

Methods of Retrofitting

3. Increasing Thickness of Existing Shear Wall

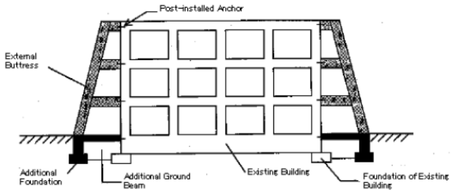


Da Vinch Ginza Building

SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

Methods of Retrofitting

7. Constructing External Buttress



SOURCE: Class note of Mr. Hiroshi OHIRA for CNCRP Project

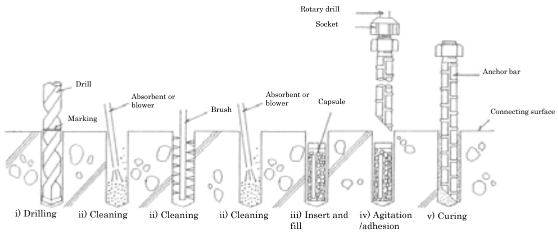
Base Isolation

Lead Rubber Bearing with Isolator used

Damper used

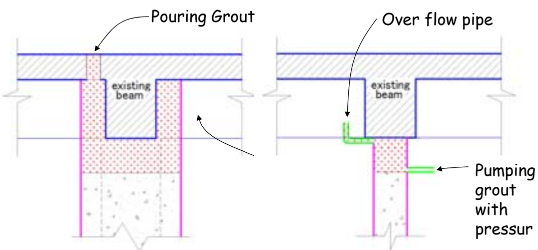
Post-Installed Anchor Work

Post-Installed Anchor Work



SOURCE: Guidelines for Seismic Retrofit of Existing Reinforced Concrete Buildings, 2001. Published by- The Japan Building Disaster Prevention Association

Non-Shrink Grout Work

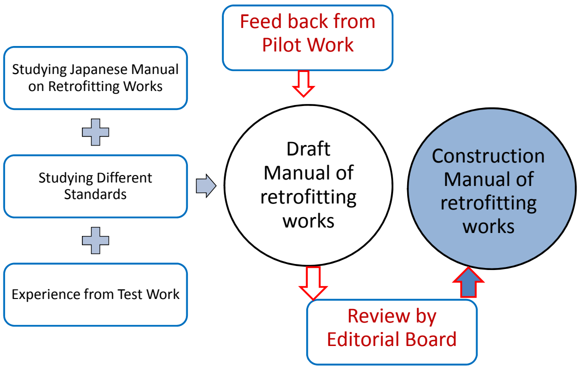


Non Shrink Grout Work Without Pressure

Non Shrink Grout Work With Pressure

Pressurized Grouting Work

Construction Manual of Retroftting works



Thank you very much

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

Short Training Course on "Techniques of Retrofit Construction for RC Buildings and Quality Control for Retrofitting Work"

Outline of Seismic Assessment and Retrofit Design

Venue: PWD
Date: 13 November, 2014

Md. Rafiqul Islam
Executive Engineer
PWD Design Division – 3
&
Team Leader
Working Team – 2
CNCRP Project

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

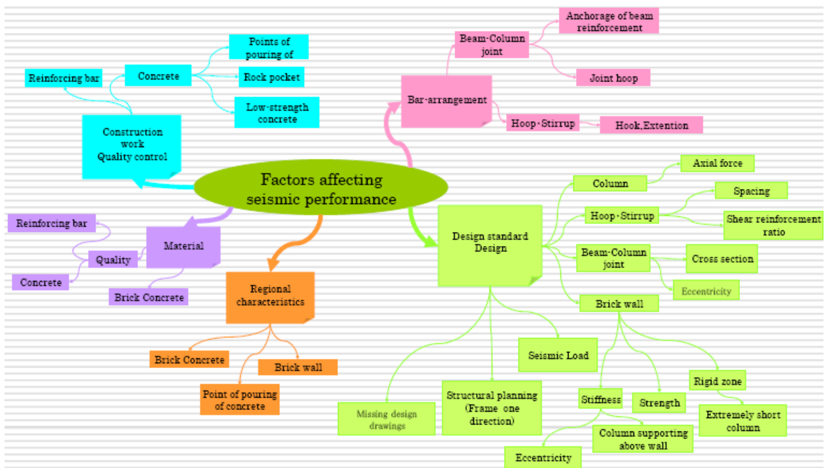
Is the Building Safe to Earthquake?

- What is the seismic intensity?
- What is the lateral load resisting system?
- What is the performance objective?
- Age of the building?
- Subsoil condition?
- Irregularity of the building?
- What is the evaluation standard?

Factors Affecting Seismic Performance

CNCRP

Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

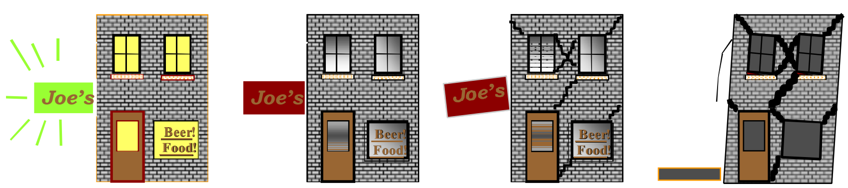


Reference: Presentation 'Issues of Seismic Performance' by Yosuke Nakajima, JICA Expert Team, 2012

Selection in Performance Based Design

CNCRP

Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings



Operational *Immediate Occupancy* *Life Safety* *Collapse Prevention*

Operational – negligible impact on building and it is fully operable

Immediate Occupancy – building is safe to occupy and retain its pre-earthquake strength and stiffness

Life Safety – building is safe during event but possibly not afterward

Collapse Prevention – building is on verge of collapse, probable total loss

Performance Level for Evaluation

Performance level checked for both structural and non-structural components:

1. Life Safety (LS) Performance Level:
   - Partial or total structural collapse does not occur
   - Damage to non-structural components is non-life-threatening
2. Immediate Occupancy (IO) Performance Level:
   - Vertical and lateral force resisting system retain nearly pre-earthquake strength
   - The damage is repairable while the building is occupied

Geologic site hazard and foundation hazard are also assessed.

Methods of Seismic Vulnerability Assessment

1. Rapid Visual Screening (FEMA 154) [Pre evaluation sage]
2. Seismic Evaluation of Existing Building (ASCE/SEI 31-03)
   - Tier 1 – Screening phase
   - Tier 2 – Evaluation phase
   - Tier 3 – Detailed evaluation phase
3. Seismic Evaluation of Existing Reinforced Concrete Structure, 2001 [Japanese standard]
4. Euro Code 8: Part 1-4
5. Guidelines for Seismic Evaluation of Existing Buildings – National Research Council Canada
6. Assessment and Improvement of the Structural Performance of Buildings in Earthquakes by New Zealand Society for Earthquake Engineering
7. Handbook on Seismic Retrofit of Buildings of India

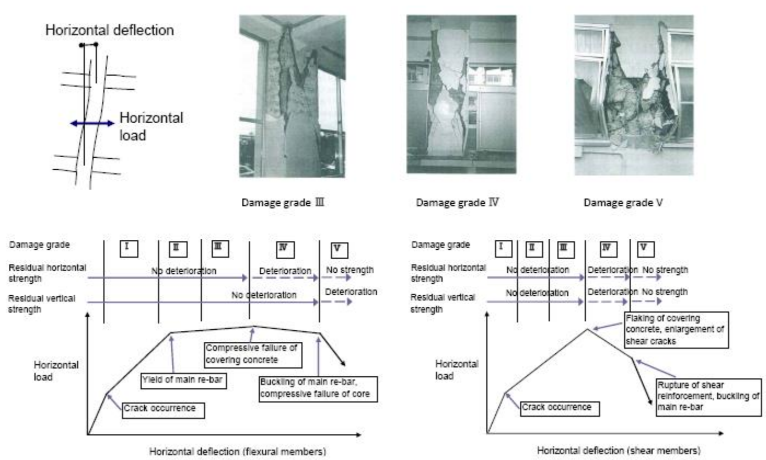
Three levels of screening in Japanese Standard

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

1. First level screening
   - Beam is extremely rigid and only vertical member will deform
   - Vertical members are classified into three categories
   - Concrete strength and sectional area of vertical member are required for calculation; reinforcement details are not required
   - Suitable for building that have too many walls
2. Second level screening
   - Beam is extremely rigid and only vertical member will deform
   - Vertical members are classified into five categories
   - Reinforcement details of vertical member is required for calculation
3. Third level screening
   - Beam is flexible and hinge may form either in beam or column
   - Vertical and horizontal members are classified into eight categories
   - Calculation process is very rigorous

Horizontal Load-Deflection Curve of RC Columns in Japan

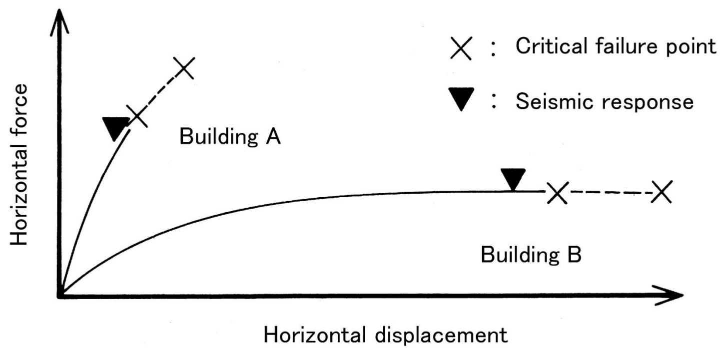
CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings



Reference: Lecture 'Seismic damages and performance of building' by Akira Inoue , JICA Expert Team, 07/06/2011

[Source: "Standard of Judgment of Damage Grade and Guidelines of Recovery Engineering for Damaged Buildings, 2001", The Japan Building Disaster Prevention Association (written in Japanese)]

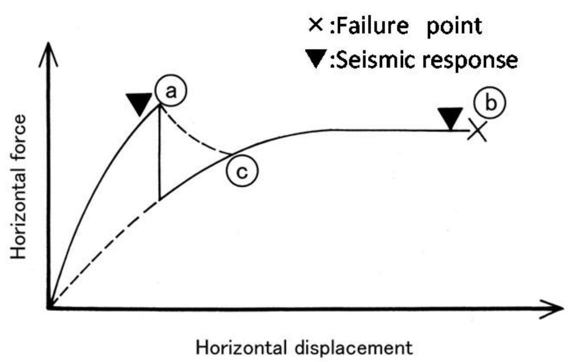
Different Types of Structure



Building 'A' resist earthquake by Strength

Building 'B' resist earthquake by Ductility

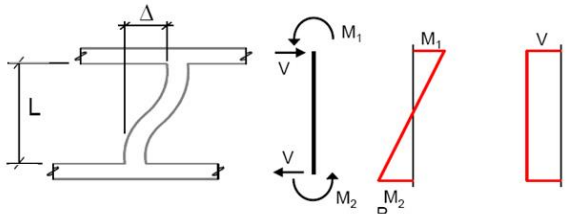
Structure Configuring Members with Different Ductility



Brittle failure of non-ductile member at 'a'
Sudden drop of stiffness from 'a' to 'c'
Performance of ductile member up to 'b'

What is Strength?

A) Flexural strength (Q_{mu}) from moment capacity of the structural member



$$Q_{mu} = \frac{M_1 + M_2}{L}$$

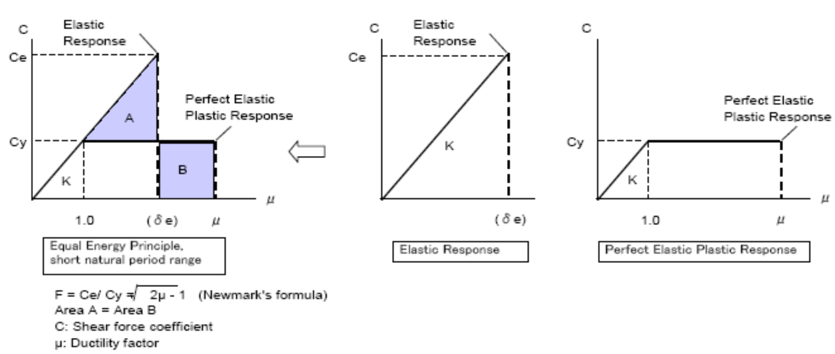
B) Shear strength (Q_{su}) from shear reinforcement of the structural member

Strength is minimum of Q_{mu} and Q_{su}

What is Ductility?

Ductility is the capacity of building material, systems or structure to absorb energy by deforming into inelastic range.

Equal Energy Principle: Ideal Non-linear Earthquake Response
In a relatively short range period building



Ref: 'Seismic Design, Evaluation and Retrofitting of Building ' by Akira Inoue, JICA Expert Team

Japanese Method of Seismic Vulnerability Assessment

Seismic index of structure $I_s = E_0 \times S_D \times T$

E_0 = Basic seismic index of structure = C x F

C = Strength index = $\frac{Q_u}{W}$

Q_u = Shear strength

W = Weight on vertical member

F = Ductility index (it is function of ductility factor)

S_D = Irregularity index

T = Time index

Seismic Demand Index

Seismic demand index, $I_{so} = E_s \times Z \times G \times U$

E_s = Basic seismic demand index (depends on level of screening)

Z = Zone index (factor for seismic intensity of the site)

G = Ground index (factor consider sub-soil condition)

U = Usage index (factor for occupancy type)

Judgment on Seismic Safety

A safe structure shall satisfy both of the following checking:

A) $I_s \geq I_{so}$

I_s = Seismic index of structure

I_{so} = Seismic demand index

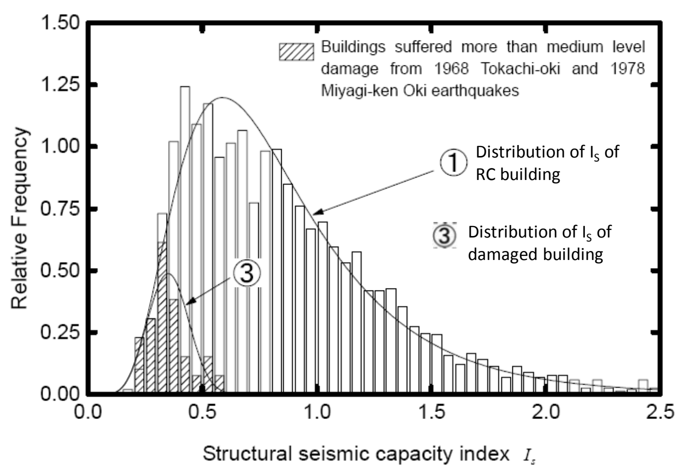
B) $C_{TU} \geq 0.3(?) \times Z \times G \times U$

C_{TU} = Cumulative strength at ultimate deformation of structure

Judgment to be applied

- Each story
- Each principal horizontal direction of a building

Study on Earthquake Damaged Buildings



Ref: Nakano, Yoshiaki and Tsuneo Okada "Reliability analysis on seismic capacity of existing reinforced concrete buildings in Japan" Journal, Transaction of Architecture Institute of Japan, No. 406, 37 – 43 (1988)

I_{SO} for Bangladeshi Buildings

Base shear, $V = \frac{ZIC}{R} W$

Z = Zone coefficient
I = Importance factor
C = Numerical coefficient for sub-soil and structural period (maximum value is 2.75)

Rearranging

$$\frac{V}{W} \times R = Z \times I \times C$$

i.e Strength index × ductility index = Z × I × C

For Dhaka Z × I × C = 0.15 × 1 × 2.75 = 0.413 (?)

Reference: Presentation 'proposed seismic demand index of structures, Iso, for existing RC buildings in Bangladesh' by Akira Inoue , JICA Expert Team, 2012

How to Calculate Strength Index (C)

Strength index, C $= \frac{Q_u}{\Sigma W}$

where ,
Q_u =Ultimate lateral load carrying capacity of vertical member
ΣW = Weight of the building supported by the story concerned

Strength index shall be modified for each story by

Story shear modification factor $= \frac{n+1}{n+i}$

n = Number of stories of a building
i = Number of story is being evaluated

How to Calculate Ductility Index (F)

Various types of deflection angle (R_{max}, R_y, R_{su}, R_{mu}, R_{mp}) of column is calculated based on:

1) Column size
2) Clear height of column
3) Axial force ratio
4) Shear force ratio
5) Tensile reinforcement ratio
6) Spacing of shear reinforcement
7) Margin against shear failure
8) etc.

How to Calculate Ductility Index (F)

A) Ductility index for shear column:

$$F = 1.0 + 0.27\frac{R_{su} - R_{250}}{R_y - R_{250}}$$

B) Ductility index for flexural column:

(i) In case $R_{mu} < R_y$

$$F = 1.0 + 0.27\frac{R_{mu} - R_{250}}{R_y - R_{250}}$$

(ii) In case $R_{mu} \geq R_y$

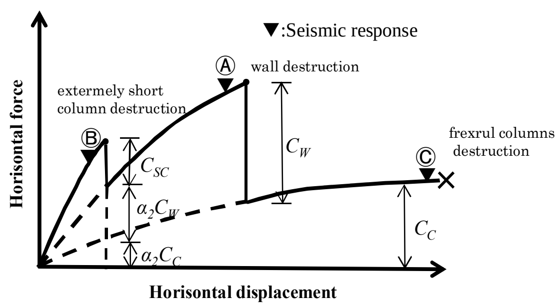
$$F = \frac{\sqrt{2\,R_{mu}/R_y - 1}}{0.75 \cdot \left(1 + 0.05\,R_{mu}/R_y\right)} \leq 3.2$$

Strength Dominant Basic Seismic Index (E_0)

$$E_0 = \frac{n+1}{n+i}\left(C_i + \sum_j \alpha_j C_j\right) \cdot F_1$$

where:

α_j= Effective strength factor



Ductility Dominant Basic Seismic Index (E_0)

$$E_0 = \frac{n+1}{n+i}\sqrt{{E_1}^2 + {E_2}^2 + {E_3}^2}$$

where:

$E_1 = C_1 \times F_1$
$E_2 = C_2 \times F_2$
$E_3 = C_3 \times F_3$

C_1 = The strength index C of the first group (with small F index).
C_2 = The strength index C of the second group (with medium F index).
C_3 = The strength index C of the third group (with large F index).
F_1 = The ductility index F of the first group.
F_2 = The ductility index F of the second group.
F_3 = The ductility index F of the third group.

Irregularity Index (S_D)

Irregularity Index covers:

1. Regularity
2. Aspect ratio of plan
3. Expansion joint
4. Well-style area
5. Underground floor
6. Story height uniformity
7. Soft story
8. Eccentricity
9. Stiffness/mass ratio

Irregularity Index (S_D) $\leq$ 1.0

Time Index (T)

Time Index evaluates:

1. Deflection of beam and column
2. Cracking in walls
3. Fire experience
4. Occupied by chemical
5. Age of building
6. Finishing condition

Time Index (T) $\leq$ 1.0

Building Inspection

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

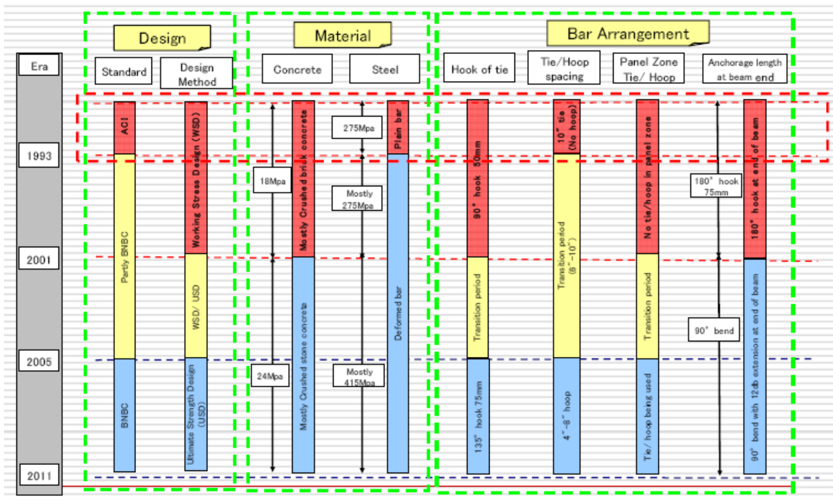
Inspection Types	Inspection Objectives	Inspection Items
Preliminary inspection	To determine the applicability of the evaluation standard	Summery of the structure and building condition
Inspection without design drawings	To inspect various structural elements by conducting the actual measurement	The dimensions of building frames and reinforcing bars, arrangement of bars, etc
Detailed inspection	▪To calculate Time index and irregularity index ▪To inspect the necessity of refurbishment of aged deterioration ▪To determine the present strength related data to enhance the accuracy of evaluation procedure	Differences from original design drawings, structural cracks, deformations. Inspect material strength, concrete neutralization depth, reinforcing bar strength etc.

Benchmark for Buildings of USA

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

	Model Building Seismic Design Provisions							
Building Type[1, 2]	NBC[ls]	SBC[ls]	UBC[ls]	IBC[ls]	NEHRP[ls]	FEMA 178[ls]	FEMA 310[ls, io]	CBC[io]
Wood Frame, Wood Shear Panels (Type W1 & W2)	1993	1994	1976	2000	1985	*	1998	1973
Wood Frame, Wood Shear Panels (Type W1A)	*	*	1997	2000	1997	*	1998	1973
Steel Moment-Resisting Frame (Type S1 & S1A)	*	*	1994[4]	2000	**	*	1998	1995
Steel Braced Frame (Type S2 & S2A)	1993	1994	1988	2000	1991	1992	1998	1973
Light Metal Frame (Type S3)	*	*	*	2000	*	1992	1998	1973
Steel Frame w/ Concrete Shear Walls (Type S4)	1993	1994	1976	2000	1985	1992	1998	1973
Reinforced Concrete Moment-Resisting Frame (Type C1)[3]	1993	1994	1976	2000	1985	*	1998	1973
Reinforced Concrete Shear Walls (Type C2 & C2A)	1993	1994	1976	2000	1985	*	1998	1973
Steel Frame with URM Infill (Type S5, S5A)	*	*	*	2000	*	*	1998	*
Concrete Frame with URM Infill (Type C3 & C3A)	*	*	*	2000	*	*	1998	*
Tilt-up Concrete (Type PC1 & PC1A)	*	*	1997	2000	*	*	1998	*
Precast Concrete Frame (Type PC2 & PC2A)	*	*	*	2000	*	1992	1998	1973
Reinforced Masonry (Type RM1)	*	*	1997	2000	*	*	1998	*
Reinforced Masonry (Type RM2)	1993	1994	1976	2000	1985	*	1998	*
Unreinforced Masonry (Type URM)[5]	*	*	1991[6]	2000	*	1992	*	*
Unreinforced Masonry (Type URMA)	*	*	*	2000	*	*	1998	*

No Benchmark for Buildings in Bangladesh

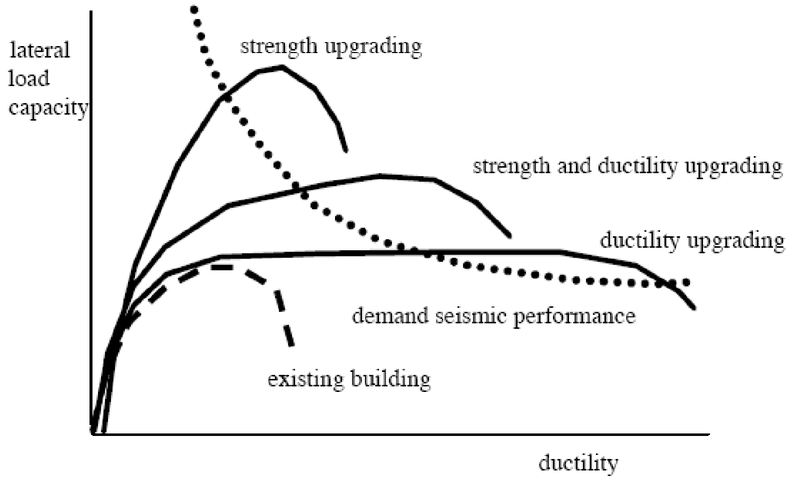


Reference: 'Issues of Seismic Performance' by Yosuke Nakajima, JICA Expert Team

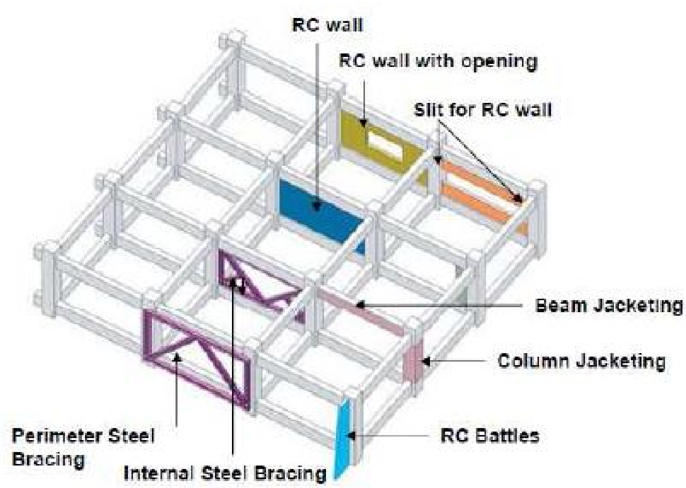
Difficulties of Seismic Assessment of Bangladeshi Buildings

1. Missing architectural and structural design of existing building.
2. Lack of reliability in construction even if drawing is available.
3. A few or no study about lateral load resisting system of building of our country.
4. Effect of infill masonry wall in frame structure.
5. Performance of mixed type (masonry + RC frame) structure.
6. Reinforcing bars of existing structure are significantly corroded.
7. Etc.

Retrofitting Objectives



Methods of Retrofitting



Methods of Retrofitting

CNCRP
Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

Types of Retrofitting Methods

No.	Description of Retrofitting Methods	Improvement of Strength	Improvement of Ductility	Improvement of Structural Balance
1	Steel Framed Bracing	○		
2	Infilling New RC Shear Wall into Open Frame	○		○
3	Increasing Thickness of Existing Shear Wall	○		○
4	Infilling Steel Plate Wall into Open Frame	○		○
5	Constructing New RC Wing Wall to RC Column	○		
6	Constructing External Frame	○		
7	Constructing External Buttress	○		
8	Steel Plate Jacketing around RC Column		○	
9	Carbon Fiber (Sheet / Strand) Wrapping around RC Column		○	
10	Concrete Jacketing around RC Column	○	○	
11	Providing New Seismic Slit		○	○

Floor Plan of Model Case

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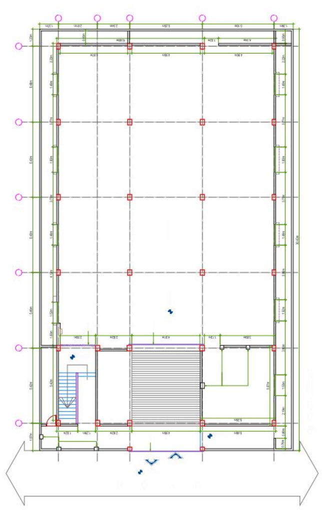
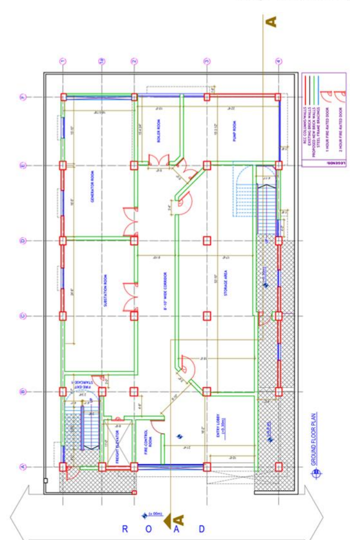


GROUND FLOOR (BEFORE)



GROUND FLOOR (AFTER)

First Floor Plan of Model Case

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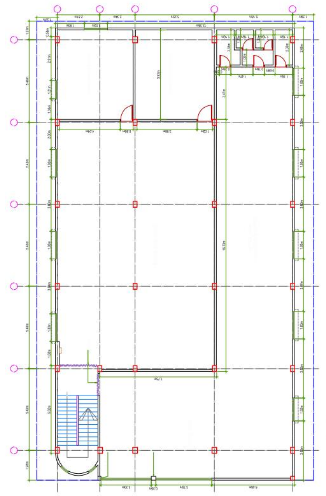
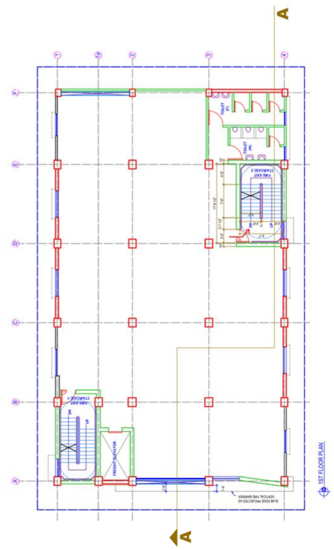


GROUND FLOOR (BEFORE)



FIRST FLOOR (AFTER)

Front Elevation of Model Case

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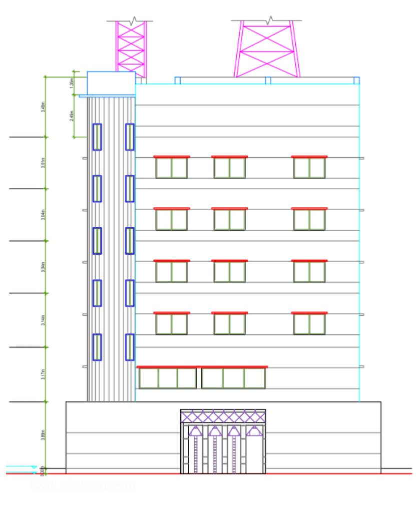
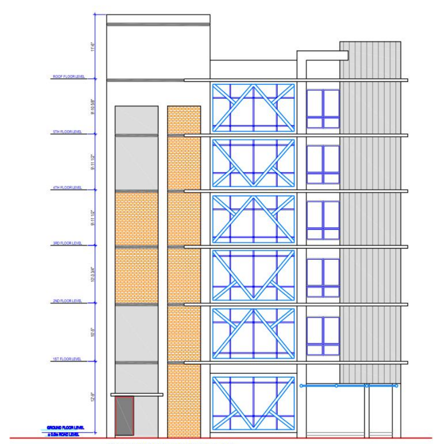


GROUND FLOOR (BEFORE)



FIRST FLOOR (AFTER)

Some Example of Retrofitting

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Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings

NIER 国立教育政策研究所文教施設研究センター
National Institute for Educational Policy Research / Educational Facilities Research Center

Factors to be Considered during Retrofit design

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- ❑ Minimum disturbance of existing floor plan.
- ❑ Optimize Cost among alternatives.
- ❑ Minimum interruption of existing function.
- ❑ Feasibility of work procedure.

Why Retrofitting is Required

- Building constructed before 1993 not following Code
- Building constructed after 1993 not considering earthquake load
- Building designed following Code but not implemented properly at field level
- Building designed and constructed following Code but retrofitting is required due to change in updated Codal requirement

QUALITY CONTROL OF RETROFIT WORKS

ABDULLAH MOHAMMOD ZUBAIR
SUB-DIVISIONAL ENGINEER
DESIGN DIV-5, DHAKA

PRESENTATION OUTLINE

- Concept of Quality control.
- Concept of Retrofitting.
- Steps to Retrofitting
- Quality Control
- Check Lists
- Photographs

Quality control

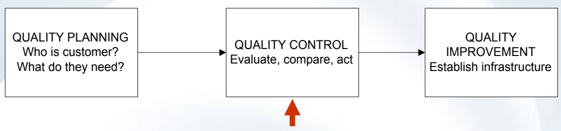
- It is a procedure or set of procedures intended to ensure that a manufactured product or performed service adheres to a defined set of quality criteria or meets the requirements of the client or customer.

Concept of Retrofitting

- Retrofitting is technical interventions in structural system of a building that improve the Resistance to earthquake by optimizing the strength, ductility and earthquake loads. Strength of the building is generated from the structural dimensions, materials, shape, and number of structural elements, etc. Ductility of the building is generated from good detailing, materials used, degree of seismic resistant, etc.

Steps to Retrofitting

- Step 1. Baseline assessment
- Step 2. Retrofit action plan
- Step 3. Undertake retrofit works
- Step 4. Complete works
- Step 5. Final assessment



QUALITY CONTROL

Inspection
- Destructive test
- Non-destructive test

Process Control
- Monitoring Process (relates to inspection, using check list, work program)
- Feedback Control

Correction
- Knowing what to correct when process is out of control

- Work program sheet (Weekly, Monthly)

- Check List item
  - Administrative & technical approval.
  - Safety and demolishing work.
  - Equipment and material's preparation
  - Excavation work.
  - Fabrication & placement of re-bar with anchorage.
  - Shuttering work.
  - Concreting work (mixing & casting).

Weekly work program

PROJECT NAME:

WEEKLY WORK PROGRAM FROM ____________ to ____________

DATE	WORK TO BE DONE	WORK DONE	REMAINING WORK	COMMENTS
	1) 2)			
	1) 2)			
	1) 2)			
	1) 2)			
	1) 2)			
	1) 2)			
	1) 2)			

Countersigned by SDE

CONTRACTOR

INSPECTION SHEET FOR ADMINISTRATIVE AND TECHNICAL APPROVAL. **SL NO: IS(AT)-**

NAME OF THE PROJECT : ____________

LOCATION OF THE PROJECT: ____________

DATE: ____________

1) Administrative approval documents like DPP and other relavant papers are present.	YES	NO	N/A
2) Notification of award is present.	YES	NO	N/A
3) Bill of quantity as per tender is present.	YES	NO	N/A
4) Work program schedule is present.	YES	NO	N/A
5) As built detail drawings are present.	YES	NO	N/A
6) Retrofit detail design drawings are present.	YES	NO	N/A
7) Concrete core cutting results are present.	YES	NO	N/A
8) Approved site plan is present.	YES	NO	N/A
9) Temporary labor shed is indicated and adequate.	YES	NO	N/A
10) Location of water supply arrangement is indicated and adequate.	YES	NO	N/A
11) Location of material storing place is idicated and adequate.	YES	NO	N/A
12) Critical distance from existing building is considered.	YES	NO	N/A
13) Electric over head line passing over the site is considered.	YES	NO	N/A
14) Trees and houses to be removed from site are considered.	YES	NO	N/A
15) Site is cleaned, cleared from temporary obstacles.	YES	NO	N/A
16) Photographs are taken	YES	NO	N/A
ARCHITECTURAL AND STRUCTURAL DRAWINGS			
17) Approved detail architectural drawings are kept in the site.	YES	NO	N/A
18) Approved detail structural drawings are kept in the site.	YES	NO	N/A
19) Architectural drawing and Structural drawing are coinciding each other (no difference) .	YES	NO	N/A

CONTRACTOR SUB-ASSISTANT ENGINEER SUB-DIVISIONAL ENGINEER

INSPECTION SHEET FOR SAFETY AND DEMOLISHING WORK **SL NO: IS(SD)-**

NAME OF THE PROJECT : ____________

LOCATION OF THE PROJECT: ____________

DATE: ____________

LOCATION OF WOEK

SAFETY			
1) Sufficient helmet are present.	YES	NO	N/A
2) Sufficient gloves are present.	YES	NO	N/A
3) Sufficient shoes are present.	YES	NO	N/A
4) Safety belt for fall protections are present.	YES	NO	N/A
5) First aid box is present & well equipped.	YES	NO	N/A
6) Proper supply of pure dinking water.	YES	NO	N/A
7) Safety net is present.	YES	NO	N/A
8) Safety canopy is present.	YES	NO	N/A
14) Warning signboards instructions for safety operation are present.	YES	NO	N/A
15) Scaffolds are erected on solid supports with handrails and tied with fixed structure.	YES	NO	N/A
16) Ladders are inspected and in good condition.	YES	NO	N/A
17) Fixed ladders in excess of 15 feet have extra protection.	YES	NO	N/A
18) Ropes and cables are in good condition.	YES	NO	N/A
19) Excavation work is barricaded and lightings are provided.	YES	NO	N/A
20) Power lines are deactivated, removed, or posted with warning signs.	YES	NO	N/A
21) At least 25 feet clearance from overhead power lines are secured.	YES	NO	N/A
22) Fire protection arrangement has done.			
23) Photographs are taken			
FOR DEMOLISHING WORK			
24) Engineering survey reports are prepared properly.	YES	NO	N/A
25) Chutes are provided for disposing of materials.	YES	NO	N/A
26) Demolishing plan is accepted by concern engineer.	YES	NO	N/A
27) Utilities like gas, water and electricity lines are disconnected.	YES	NO	N/A
28) Hazardous materials/chemicals are removed from any pipes, tanks, or equipment.	YES	NO	N/A
29) Temporary support has been provided around the area to be demolished.	YES	NO	N/A
30) All signs, warning signals, and protective equipment are in place.	YES	NO	N/A
31) Photographs are taken	YES	NO	N/A

INSPECTION SHEET FOR EXCAVATION **SL NO: IS(EX)-**

NAME OF THE PROJECT : ____________

LOCATION OF THE PROJECT: ____________

DATE: ____________

1) As built site plan is present.	YES	NO	N/A
2) R.L (Reduced level) Sheet is presented before excavation	YES	NO	N/A
3) Earth cutting location & variable depth is considered.	YES	NO	N/A
4) Protection methods of soil are considered.	YES	NO	N/A
5) Decomposed layer is indicated with sign.	YES	NO	N/A
6) Excavation is executed to the target depth.	YES	NO	N/A
7) Foundation bed is properly compacted and level.	YES	NO	N/A
8) Sand over the foundation bed is compacted well.	YES	NO	N/A
9) Khoa -sand over the foundation bed is compacted well.	YES	NO	N/A
10) Any objectionable observations in foundation bed are removed.	YES	NO	N/A
11) Existing utility lines (Gas, Sewerage, Electric, Supply water, Etc) are considered	YES	NO	N/A
12) Existing hazards structure are considered. (suface drain,pit,piller etc.)	YES	NO	N/A
13) Provision for dewatering is considered	YES	NO	N/A
14) Rain protection during excavation is considered.	YES	NO	N/A
15) Earth stacking yard is considered	YES	NO	N/A
16) Photographs are taken	YES	NO	N/A

CONTRACTOR SUB-ASSISTANT ENGINEER SUB-DIVISIONAL ENGINEER

INSPECTION SHEET FOR EQUIPMENT AND MATERIAL'S PREPRATION **SL NO: IS(EM)-**

NAME OF THE PROJECT : ____________ DATE: ____________

LOCATION OF THE PROJECT: ____________

MATERIALS			
1) Aggregates are free from dust	YES	NO	N/A
2) Aggregates size is as per specification.	YES	NO	N/A
3) Brick aggregates are well burnt.	YES	NO	N/A
4) Aggregates are well graded.	YES	NO	N/A
5) Aggregates are stored on solid platform.	YES	NO	N/A
6) F.M of sand is as per specification.	YES	NO	N/A
7) Sand is free from dust & other impurities	YES	NO	N/A
8) Sand is stored on cleaned solid platform	YES	NO	N/A
9) Types of cement (CEIM-1 , CEIM-2) is satisfied as per specification.	YES	NO	N/A
10) Cement bags are properly stored.	YES	NO	N/A
11) Admixture as per design specification is present.	YES	NO	N/A
12) Availability of portable water.	YES	NO	N/A
13) Reinforcement as per design specification (Strength, Size,Brand).	YES	NO	N/A
14) Reinforcement are proprly stored according to Engineer instruction.	YES	NO	N/A
15) Carbon fiber as per design specification are present.	YES	NO	N/A
16) Adhesive for carbon fiber raping are present.	YES	NO	N/A
17) Epoxy Adhesive are present.	YES	NO	N/A
EQUIPMENTS			
18) Mixure machine is provided.	YES	NO	N/A
19) Standard size of steel buckets are provided, to carry aggregates.	YES	NO	N/A
20) Vibrator machines are provided.	YES	NO	N/A
21) Vibrator spare nozzles are present	YES	NO	N/A
22) Levelling instruments are present.	YES	NO	N/A
23) Equipments are tested before the comencement of works and well condition.	YES	NO	N/A
24) Roof/ Tower hoist machine is provided.	YES	NO	N/A
25) Cylender mold are present as per requirement	YES	NO	N/A
26) Slump cone is present.	YES	NO	N/A
27) Drill machine, having proper dia of dril bit's are present.	YES	NO	N/A
28) Available power line or generator power system.	YES	NO	N/A
29) Special type pump machine for casting none shrink micro concrete are present.	YES	NO	N/A
30) Epoxy Adhesive injector's for injecting adhesive inside narrow hole, are present.	YES	NO	N/A
31) Photographs are taken	YES	NO	N/A

CONTRACTOR SUB-ASSISTANT ENGINEER SUB-DIVISIONAL ENGINEER

INSPECTION SHEET FOR FABRICATION & PLACEMENT OF RE-BAR WITH ANCHORAGE **SL NO: IS(RC)-**

A NAME OF THE PROJECT : ____________

B LOCATION OF THE PROJECT: ____________

C DATE: ____________

D FABRICATION & PLACEMENT OF REBAR LOCATION

E SIZE OF RE-BAR 10 mm | 12 mm | 16 mm | 20 mm | 25 mm

G RE-BAR GRADE & RE-BAR SCHEDULE

FABRICATION & PLACEMENT OF RE-BAR			
1) Approved re-bar test report's are present	YES	NO	N/A
2) Re-bar strength (grade) are as per design specifications.	YES	NO	N/A
3) Size (length and diameter) of re-bar are as per design.	YES	NO	N/A
4) Hooks and bends are provided as per design specification.	YES	NO	N/A
5) Re-bar is free from any crack.	YES	NO	N/A
6) Number of re-bar are as per design.	YES	NO	N/A
7) Spacing of re-bar are as per design.	YES	NO	N/A
8) Main re-bar dirction are as per design instruction.	YES	NO	N/A
9) Re-bar is tied with 24 SWG wire (in each and every joint)	YES	NO	N/A
10) Splice length is as per design specification(tension & compresion)	YES	NO	N/A
11) Standard sciesmic hook is provided.	YES	NO	N/A
12) Alignment of rebar is as per drawing's	YES	NO	N/A
13) Clear cover is set properly as per design specification	YES	NO	N/A
14) Re-bar is cleaned from all loose scale, dirt, and paint oil, grease or other foreign substance.	YES	NO	N/A
15) Location of splice is at the middle of column	YES	NO	N/A
16) Tie spacing is maintained as per design specification	YES	NO	N/A
17) Seperator is provided.	YES	NO	N/A
18) Corner reinforcement is provided	YES	NO	N/A
19) Extra top bar is provided	YES	NO	N/A
20) Closer tie spacing near splice zone is as per design.	YES	NO	N/A
21) Inner tie is provided as per design.	YES	NO	N/A
22) Stirrup is privided as per design.	YES	NO	N/A
23) Plumding or electrical duct/punch is provided as per drawing.	YES	NO	N/A

24) Electrical conduit is placed as per drawing.	YES	NO	N/A
ANCHORAGE			
25) Drill location's are confirm as per design	YES	NO	N/A
26) Drill depth & diameter are maintained as per design	YES	NO	N/A
27) Drilled hole's are properly cleaned	YES	NO	N/A
28) Epoxy adhesive is properly inject into the drilled hole.	YES	NO	N/A
29) Anchorage bolt/ re-bar are set properly.	YES	NO	N/A
30) Curing is done as specification	YES	NO	N/A
31) Location of pull out test is indicated.	YES	NO	N/A
32) Photograph's are taken	YES	NO	N/A

CONTRACTOR — SUB-ASSISTANT ENGINEER — SUB-DIVISIONAL ENGINEER

INSPECTION SHEET FOR SHUTTERING WORK (Column). **SL NO: IS(SC)-**

A) NAME OF THE PROJECT :

B) LOCATION OF THE PROJECT:

C) DATE: G) TYPES OF SHUTTER: SINGLE / DUBLE / MULTI

D) SHUTTERING MEMBER'S: FOOTING / COLUMN / BEAM / SLAB

E) LOCATION OF THE MEMBER'S

F) REBAR CHECK LIST REFERANCE NUMBER

1) Re-bar check list is cosidered	YES	NO	N/A
2) Quality of shuttering materials are approved by site in charge.	YES	NO	N/A
3) Joints of wooden shutter are covered with thin ci sheet piece.	YES	NO	N/A
4) Forms are cleaned with steel scraper or steel brush to make it free	YES	NO	N/A
from rust, mortar encrustations or other foreign material.	YES	NO	N/A
5) Shutter oil is applied properly	YES	NO	N/A
6) Lateral bracing is provided	YES	NO	N/A
7) Shutter joints are properly fixed rubber gasket.	YES	NO	N/A
8) Shutter surface are proprly leak proof.	YES	NO	N/A
9) Shutter supports are at both end fixed proprly	YES	NO	N/A
10) Props at both ends are tighten.	YES	NO	N/A
11) Loose props/defective props are removed.	YES	NO	N/A
12) Scaffolding is set for safety where it is necessary.	YES	NO	N/A
13) When shuttering height exceed 12 feet, special type of scaffolding	YES	NO	N/A
with cross bracing is used.	YES	NO	N/A
14) Shuttering nuts and bolt are fixed properly.	YES	NO	N/A
15) Electric conduits / sanitary duct are set properly before casting.	YES	NO	N/A
16) Clear cover is set properly	YES	NO	N/A
17) Polythine not used in shuttering work.	YES	NO	N/A
18) Proper vertical alignment of shuttering is maintained.	YES	NO	N/A
19) Uniform lateral supports are ensured.	YES	NO	N/A
20) Photograph's are taken	YES	NO	N/A

CONTRACTOR — SUB-ASSISTANT ENGINEER — SUB-DIVISIONAL ENGINEER

INSPECTION SHEET FOR CONCRETING WORK (MIXING & CASTING) **SL NO: IS(MC)-**

A) NAME OF THE PROJECT :

B) LOCATION OF THE PROJECT:

C) DATE:

D) CONCRETING MEMBER: FOOTING / COLUMN / BEAM / SLAB

E) CASTING LOCATION

F) REFERANCE NO OF INSPECTION SHEET FOR EQUIPMENT AND MATERIAL'S PREPRATION.

G) REFERANCE NO OF INSPECTION SHEET FOR REINFORCEMENT WORK .

1) Inspection sheet for equipment and material's prepration is checked	YES	NO	N/A
2) Inspection sheet for reinforcement work is checked	YES	NO	N/A
3) For bonding with old concrete, the edges of old concrete are made rough to give a firm bonding.	YES	NO	N/A
4) Concrete rough surface are washed properly with clean water.	YES	NO	N/A
5) Grouting is used for construction joint.	YES	NO	N/A
6) Rain protection arrangement is considered	YES	NO	N/A
7) Casting sequence/phase are decided with engineer in charge.	YES	NO	N/A
8) Estimated volumn of concrete materials (with admixture) are present sufficiently	YES	NO	N/A
9) Enough manpower (including site engineer) are present.	YES	NO	N/A
10) At mixture machine location, specific person for load counting is deputed.	YES	NO	N/A
11) During casting sufficient shuttering worker are kept for constant watch on prop.	YES	NO	N/A
12) Concrete Mix Design/ Mix propotion is provided as per Design specification	YES	NO	N/A
13) Concled work item's are consider(Electrical, Water, Gas, ETC.)	YES	NO	N/A
14) Volume of concrete is estimated and the target time is set.	YES	NO	N/A
15) Adhesive chemical's for bonding between old and new concrete is used as per design specification.	YES	NO	N/A
16) Admixture is used in Concrete as per design sppecification (water reduceing, air entering,Water proofing Etc.)	YES	NO	N/A
17) Steel Bucket(Fara) is provided for aggregates and sand. (local name), equivalent to a bag of cement, to maintain the proportion.	YES	NO	N/A
18) Mixing portion of cement, aggregates and sand are investigated as per design specification.	YES	NO	N/A
19) Aggregates are properly washed	YES	NO	N/A
20) Water cement ratio is maintained as per Design Specification.	YES	NO	N/A

21) Workability of concrete is adequate (slump is proper)	YES	NO	N/A
22) Concrete cylinder's are taken to testing	YES	NO	N/A
23) Thickness is checked at regular interval over the concrete slab.	YES	NO	N/A
24) Compaction is done within the 15 minutes after the fresh concrete is placed in position.	YES	NO	N/A
25) Vibrating is stopped as soon as laitance of mortar appears on the surface. (excessive vibration segregates the mix.)	YES	NO	N/A
26) Vibrating nozzle is placed vertically.	YES	NO	N/A
27) Vibrating is executed without touching the re-bar.	YES	NO	N/A
28) Concrete is placed within 30 minutes after mixing the water with cement.	YES	NO	N/A
29) Height of free fall of concrete is within 3 feet is maintained.	YES	NO	N/A
30) Photograph's are taken	YES	NO	N/A

CONTRACTOR — SUB-ASSISTANT ENGINEER — SUB-DIVISIONAL ENGINEER

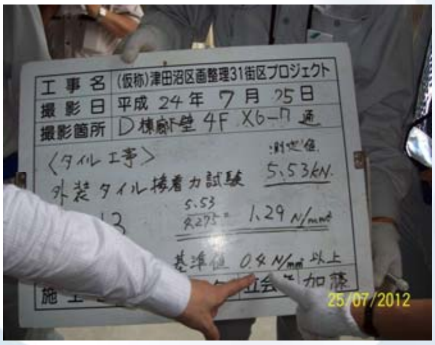
- Quality control photograph in Japan.

CNCRP

- Quality control photograph (Japan)

» Job Responsibility for each item of a room (with their name and designation)

- Quality control photograph (Japan)



Check List for various Items

- Quality control photograph (Japan)

Temporary Shed for stacking cement at field.

- Quality control photograph-Safety (Japan)

Temporary Shed for stacking cement at field.

- Quality control photograph-Cleanness (Japan)

- Quality control photograph-Cleanness (Japan)

Clean at working site and use of Helmet

- Quality control photograph-Cleanness (Japan)

Clean at working site.

- Quality control photograph-Cleanness (Japan)

Clean at working site.

Photographs :
- Our practice and problem findings.

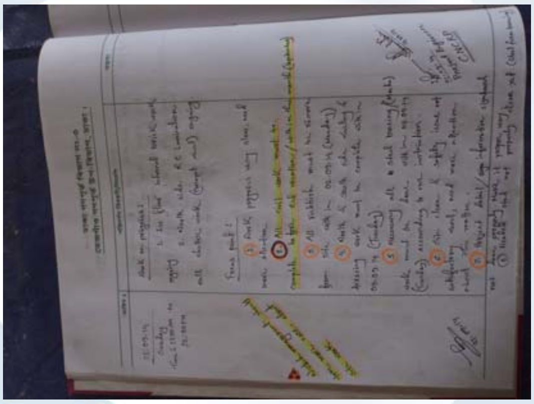
Safety awareness

Fabrication Check

Fabrication Check

Fabrication Check

Site order book

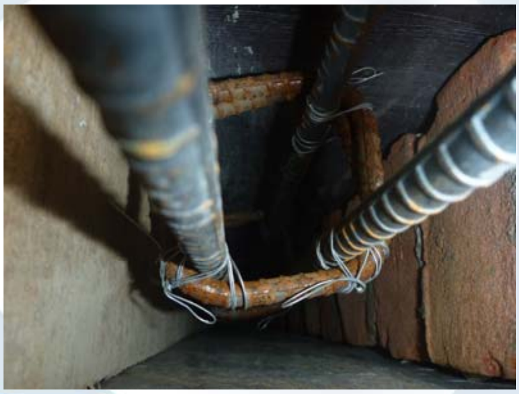


Various Component

Steel Bracing

Curing …

Problem findings .. . . .



Casting surface

Brick stacking

Brick stacking

Safety awareness

- Corrections and modifications

-: Thank You :-

Management of Retrofitting Construction

Md. Mafizur Rahman
And
Md. Sohel Rahman
CNCRP Project

Outline

- Considerations of Retrofit work
- Necessity of Test work
- Construction Management outline
- Construction Flowchart
- Controls on Documents, Materials, Performance & Safety.
- Himeji Castle: A Case Study

Retrofitting Work

What are special considerations!

A. Work Nature
B. New Technology
C. New Materials

Retrofitting Work

What are special considerations!

Restraint work condition →

- Small scale construction
- Segmented construction
- Limited access due to occupier
- Dust and Noise control
- Time constraint

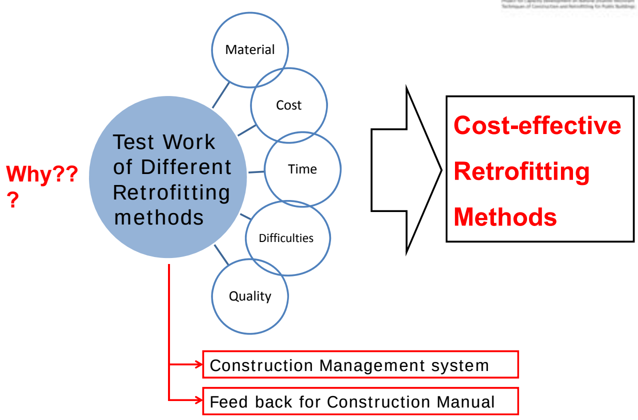
Retrofitting Work

What are special considerations!

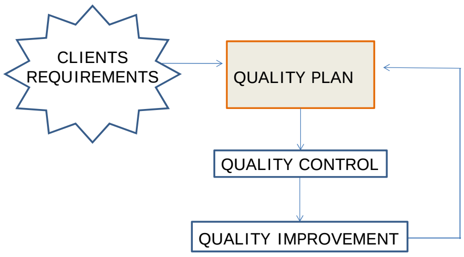
Knowing New Techniques & Materials →

- Post installed anchor work.
- Non shrink Grout Application.
- Pressure concreting
- Wrapping of carbon/glass fiber
- Use of construction chemicals to improve workability, strength etc.

Retrofitting Test Work



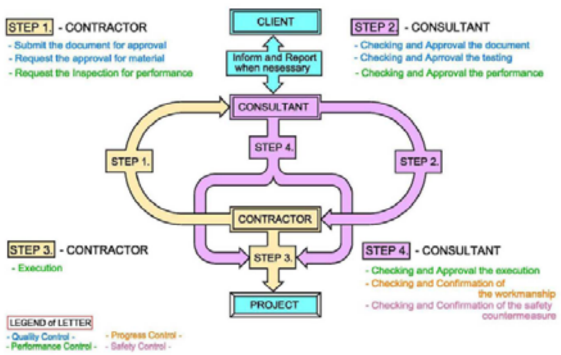
Preparation of Quality Plan



Outline of Construction Management

- Documents Control (File Management)
- Quality Control- Material
- Quality Control- Performance
- Progress Control
- Safety Control

Flow Chart : Construction Stage



Documents Control

S/V1 : Documents for commencement work 1.Commencement Order 2.Hand over letter to Contractor, etc. : Construction Management Manual - Prepared by Consultant - : Work Plan by Contractor	S/V5 : Weekly Report by Contractor 1. Minutes of Meeting 2. Weekly Progress Schedule 3. Quality Control Report, if necessary 4. Performance Control Report, if necessary 5. Safety Control Report, if necessary
S/V2 : Communication Letter 1.Question and Answer letter with Client 2.Question and Answer letter with Contractor	S/V6 : Monthly Report 1. Submit to Client , if necessary
S/V3 : Quality Control -Approved Document- 1. Materials : a. Certificate of Each Materials, Mill Sheet, Frog Mark of Brand, etc. by Contractor b. Testing Report Include Picture by PWD 2. Shop Drawings by Contractor, if possible	S/V7 : Document of Contract and Payment 1. Request Letter from Concerned Authority 2. Contract of Contractor 3. Payment Record
S/V4 : Performance Control - Inspection Report by Contractor, approved by PWD - 1. Inspection Record include Pictures – Architectural work - 2. Inspection Record include Pictures – Mechanical work - 3. Inspection Record include Pictures – Electrical work - 4. Inspection Record include Pictures – Plumbing work -	S/V8 : Completion Document 1. As- built Drawing by Contractor 2. Final Inspection Report by Consultant 3. Completion Report by Consultant

Documents Control

1 Documents for commencement Work

- Work Order
- Hand Over letter to contractor
- Management Guideline by consultant
- Work plan by contractor etc.

2 Communication letters

- Question and answer letter with client
- Question and answer letter with contractor.

Documents Control

3 Quality Control- Material approved documents

- Certificates of each materials, Mill sheet, Frog mark of brand etc. by contractor.
- Testing Reports including pictures.
- Shop drawing by contractor, if necessary.

4 Performance Control

- Inspection Report by contractor approved by the consultant/ authority
- Inspection Record include Pictures

Documents Control

5 **Weekly Reports by Contractor**

- Minutes of Meeting
- Weekly Progress Schedule
- Quality Control Report, if necessary
- Performance Control Report, if necessary
- Safety Control Report, if necessary

6 **Monthly Report**

- Monthly report submit to client.

Documents Control

7 **Documents of Contract and Payment**

- Request Letter from concerned authority
- Contract of contractor
- Payment Records

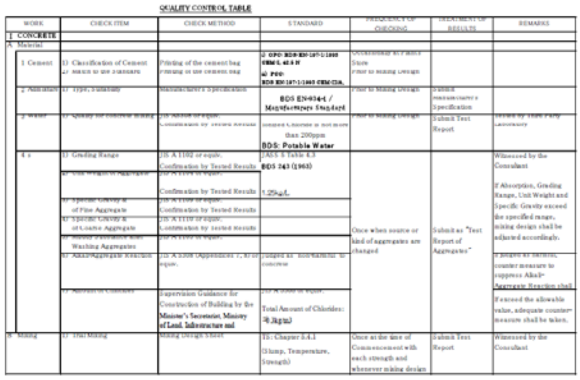
8 **Completion Documents**

- As-built drawing by Contractor
- Final Inspection Report by Consultant
- Completion Report by Consultant

Quality Control- Materials

- A Quality Control Program must be set by the consultant as per design and specification, such as Quality Control Table, Performance Control Table etc.
- Confirmation of quality of materials as per the quality control program
- Confirmation of quality of performance as per the quality control program
- Confirmation of quality of progress as per the quality control program
- Confirmation of safety of performance as per the quality control program

Quality Control- Materials


QUALITY CONTROL TABLE

WORK	CHECK ITEM	CHECK METHOD	STANDARD	FREQUENCY OF CHECKING	TREATMENT OF RESULTS	REMARKS
I CONCRETE						
A Material						
1 Cement	1) Classification of Cement 2) [illegible]	Printing of the cement bag [illegible]	i) OPC: BDS-EN-197-1:1993 CEM-I, 42.5 N ii) PCC: BDS EN 197-1:1993 CEM-II/A,	[illegible] Store [illegible]		
2 [illegible]	1) [illegible]	[illegible]	BDS EN-934-1 / Manufacturers Standard	[illegible]	Submit [illegible] Specification	
3 [illegible]	1) [illegible]	[illegible]	[illegible] than 200ppm BDS: Potable Water	[illegible]	Submit Test Report	[illegible]
4 [illegible]	1) Grading Range	JIS A 1102 or equiv. Confirmation by Tested Results	JASS 5 Table 4.3 BDS 243 (1963)	Once when source or kind of aggregates are changed	Submit as "Test Report of Aggregates"	Witnessed by the Consultant If Absorption, Grading Range, Unit Weight and Specific Gravity exceed the specified range, mixing design shall be adjusted accordingly.
	2) [illegible]	[illegible] Confirmation by Tested Results	1.25kg/L			
	3) [illegible] of Fine Aggregate	[illegible] Confirmation by Tested Results				
	4) [illegible] of Coarse Aggregate	[illegible] [illegible]				
	5) [illegible] Washing Aggregates	[illegible]				
	6) [illegible]	[illegible] equiv.	[illegible] concrete			[illegible] counter measure to suppress Alkali-Aggregate Reaction shall
	7) [illegible]	Supervision Guidance for Construction of Building by the Minister's Secretariat, Ministry of Land, Infrastructure and	[illegible] Total Amount of Chlorides: [illegible]			If exceed the allowable value, adequate counter-measure shall be taken.
[illegible]	1) [illegible]	[illegible]	TS: Chapter 5.4.1 (Slump, Temperature, Strength)	Once at the time of Commencement with each strength and whenever mixing design	Submit Test Report	Witnessed by the Consultant

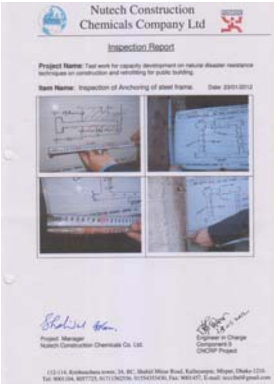
Quality Control- Materials

QUALITY CONTROL TABLE			
WORK	CHECK ITEM	CHECK METHOD	STANDARD
CONCRETE			
A Material			
1 Cement	1) Classification of Cement	Printing of the cement bag	i) OPC: BDS-EN-197-1:1993 CEM-I, 52.5 N
	2) Match to the Standard	Printing of the cement bag	
2 Admixture	1) Type, Suitability	Manufacturer's Specification	BDS EN-934-1 / Manufacturers Standard

Quality Control- Materials

	QUALITY CONTROL TABLE	(Continued)	
	FREQUENCY OF CHECKING	CONTRACTTOR'S RESPONSIBILITY	REMARKS
1 Cement	Occasionally at Plant's Store prior to mixing design	Approval to be taken before mixing	
2 Admixture	Prior to Mixing Design	Manufacturer's specification and previous test record to be submitted before mixing for approval	

Quality Control- Materials



Quality Control- Performance

- Confirmation of quality of performance as per the performance control program

Quality Control- Performance

CHECK LIST FOR THE PERFORMED WORK

WORK	MEASUREMENT ITEM	METHOD	FREQUENCY OF MEASUREMENT	UNIT mm	TREATMENT OF RESULTS	ALLOWABLE TOLERANCE	REMARKS
A Earthwork							
1 Excavation	Bottom Level	1	• 4 corner and 1 center for square trench • every 5m at center of trench • every 5m in length and breadth for overall excavation	10	Survey Record	±30 mm	Photo record shall be attached
2 Backfilling	Top Level	1	• every 5m in length and breadth	10	Survey Record	±30 mm	Photo record shall be attached
B Foundation Work							
1 Gravel /Crushed Stone	Top Level	1	• 4 corner and 1 center for square trench • every 5m at center of trench • every 5m in length and breadth for overall excavation	10	Survey Record	±30 mm	Photo record shall be attached
	Width	2	Every 5m	10	Survey Record	±50 mm	Photo record shall be attached
C Concrete Work							
1 Footing Underground Beam	Top Level	1	Random	1	Survey Record	±20 mm	Photo record shall be attached
	Cross Section Dimensions	2	Random	1	Survey Record	+50 ~ -10 mm	Photo record shall be attached
2 Slab on Grade	Top Level	1	Random	1	Survey Record	±10 mm	Photo record shall be attached
3 Column	Cross Section Dimensions	2	Random	1	Survey Record	+20 ~ -5 mm	Photo record shall be attached
	Deviation from Plumb Line	2	Random	1	Survey Record	±20 mm	Photo record shall be attached
4 Girder, Beam	Cross Section Dimensions	2	Random	1	Survey Record	+20 ~ -5 mm	Photo record shall be attached
	Bottom Level	1	Random	1	Survey Record	±20 mm	Photo record shall be attached
5 Slab	Top Level	1	Random	1	Survey Record	±10 mm	Photo record shall be attached
	Bottom Level	1	Random	1	Survey Record	±20 mm	Photo record shall be attached

Quality Control- Performance

PERFORMANCE CONTROL TABLE

WORK	MEASUREMENT ITEM	METHOD	FREQUENCY OF MEASUREMENT
D Structural Work			
1 Reinforcement Work	Diameter, number and space	As per Design Drawings	When completed assembling re-bars.
	Length and location of splice joints, Anchor length	As per Design Drawings	When completed assembling re-bars.
	Concrete coverage, Additional bar	As per Design Drawings and Specifications	When completed assembling re-bars.

Quality Control- Performance

PERFORMANCE CONTROL TABLE (Contd.)

	UNIT mm	TREATMENT	ALLOWABLE	REMARKS
		OF RESULTS	TOLERANCE	
1 Reinforcement Work		Inspection Record		Photo record shall be attached
				Photo record shall be attached
				Photo record shall be attached

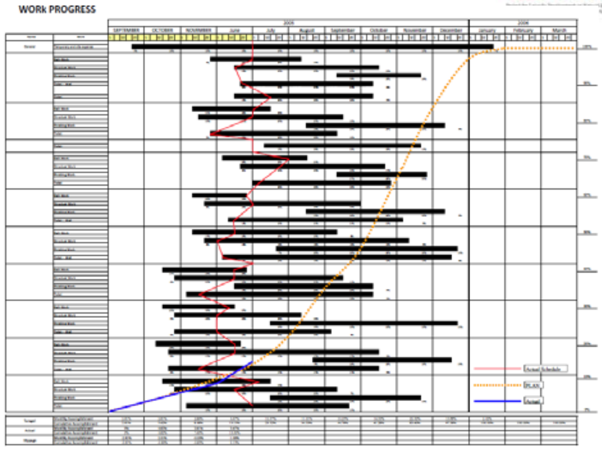
Quality Control- Performance

- Strength test of anchor by hammering
- Site: Chiba Prefectural Sakura Higashi Senior High School

Progress Control

- Request the contractor to submit the work progress chart/ graph (Time schedule).
- Confirm the work progress at the weekly meeting, on schedule or not.
- When find out the problem of progress, discuss with the contractor how to solve.

Progress Control



Safety Control

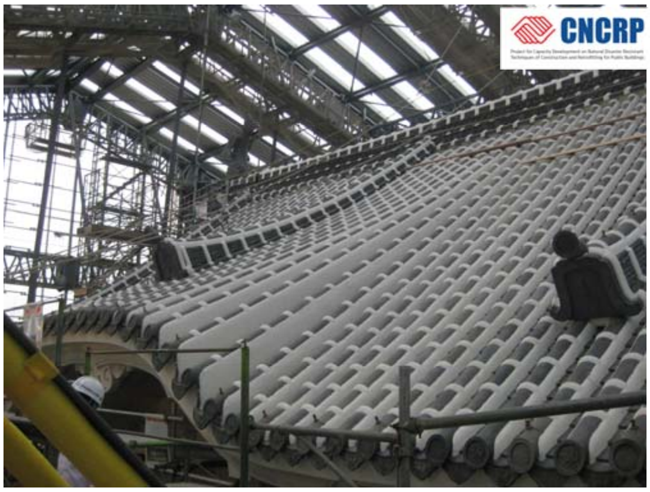
- Confirm the safety measures taken for workers and local residents.
- Make sure crisis management is foolproof and confirm the emergency response

Safety Control

Worker with Safety Harness
Site: IDEC Corporation new HQ

Worker with Safety Harness
Site: IDEC Corporation new HQ

Himeji Castle: A Case Study



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Thank you very much