SN 2007

SEMINAR NASIONAL 2007

WORKSHOP& KONTES BANGUNAN TAHAN GEMPA

27-28 APRIL 2007

KULIAH UMUM

"PERANCANGAN BANGUNAN TAHAN GEMPA"

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SESI I

PEMBICARA : DJONI SIMANTA Ir., MT.

TOPIK : PEMBAHASAN MENGENAI SNI 1726-2002 DAN SNI 2847-2002 (PASAL 23)

PUSAT DOKUMENTASI INFORMASI ILMIAH UNIVERSITAS KATOLIK PARAHYANGAN

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HIMPUNAN MAHASISWA JURUSAN SIPIL

UNIVERSITAS KATOLIK PARAHYANGAN

ANALISIS DAN DESAIN TAHAN GEMPA UNTUK RUMAH DAN GEDUNG BERDASARKAN SNI 03-1726-2002 DAN SNI 03-2847-2002

Dipersiapkan oleh
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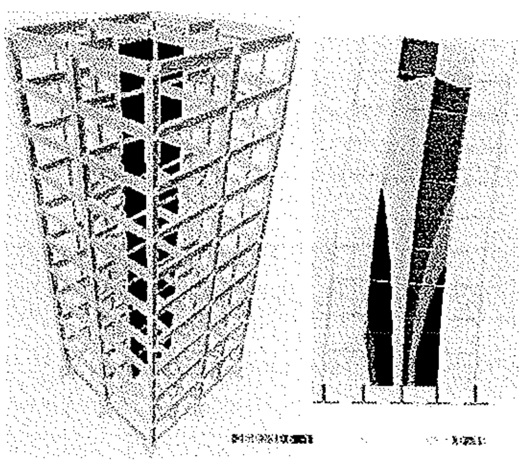
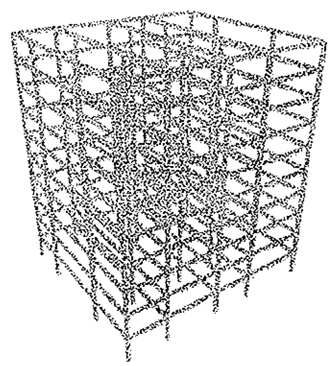
JURUSAN TEKNIK SIPIL
UNIVERSITAS KATOLIK PARAHYANGAN
BANDUNG
2007

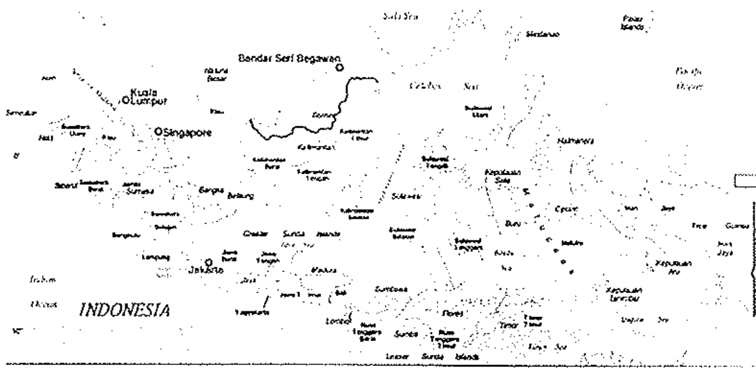
Shear Wall–Frame Interaction (Dual Systems)



Moment Resisting Frame



Kepulauan Indonesia



daerah rawan gempa

Latar Belakang

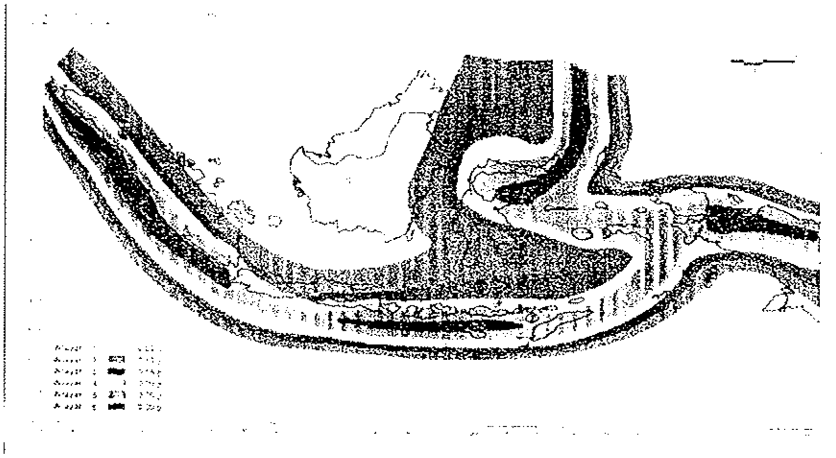
CODES

Pedoman-pedoman baru

* "Tata cara perencanaan ketahanan gempa untuk bangunan gedung SNI 03-1726-2002"
* "Tata cara perencanaan struktur beton untuk bangunan gedung SNI 03-2847-2002"

SNI 03-1726-2002

Peta Wilayah Gempa Indonesia



KEGEMPAAN INDONESIA

Peta Wilayah Gempa Indonesia

Berdasarkan Seismic Hazard Analysis, diperoleh percepatan puncak batuan dasar yang memiliki periode ulang 500 tahun di sejumlah titik lokasi di Indonesia.

Di atas peta Indonesia ditarik garis-garis kontur yang menghubungkan titik-titik lokasi yg memiliki percepatan puncak batuan dasar yang sama, sehingga diperoleh dasar dari pembuatan Peta Wilayah Gempa Indonesia.

Indonesia dibagi dalam 6 Wilayah Gempa berikut :

Wilayah	Percepatan puncak batuan dasar ('g') dengan periode ulang 500 tahun
1	0,03
2	0,10
3	0,15
4	0,20
5	0,25
6	0,30

JENIS TANAH & PERCEPATAN PUNCAK MUKA TANAH

- Gelombang gempa merambat dari batuan dasar ke muka tanah sambil mengalami perbesaran gerak, tergantung pada jenis tanah yang ada di atas batuan dasar.
- Dibedakan 3 jenis tanah: Tanah Keras, Tanah Sedang dan Tanah Lunak, ditentukan berdasarkan nilai rata-rata berbobot 3 parameter tanah, yaitu kecepatan rambat gelombang geser vs, nilai test pnetrasi standar N dan kuat geser niralir Su.
- Dengan tebal lapisan tanah t sebagai faktor pembobot, nilai rata-rata berbobot parameter tanah menjadi sebagai berikut :

$$\bar{v}_s = \frac{\sum_{i=1}^{m} t_i}{\sum_{i=1}^{m} t_i / v_{si}} \qquad \bar{N} = \frac{\sum_{i=1}^{m} t_i}{\sum_{i=1}^{m} t_i / N_i} \qquad \bar{S}_u = \frac{\sum_{i=1}^{m} t_i}{\sum_{i=1}^{m} t_i / S_{ui}}$$

Dengan syarat : $\sum_{i=1}^{m} t_i \leq 30\,m$

Karena menurut penelitian hanya lapisan-lapisan tanah sampai kedalaman 30m saja yang Menentukan pembesaran.

JENIS TANAH & PERCEPATAN PUNCAK MUKA TANAH

Tabel parameter jenis-jenis tanah

Jenis Tanah	Kecepatan rambat gelombang geser rata-rata $\bar{v}_s$ (m/det)	Nilai test penetrasi standar rata-rata $\bar{N}$	Kuat geser niralir $\bar{S}_u$ (kPa)
Tanah Keras	$\bar{v}_s \geq 350$	$\bar{N} \geq 50$	$\bar{S}_u \geq 100$
Tanah Sedang	$175 \leq \bar{v}_s < 350$	$15 \leq \bar{N} < 50$	$50 \leq \bar{S}_u < 100$
Tanah Lunak	$\bar{v}_s < 175$	$\bar{N} < 15$	$\bar{S}_u < 50$
Tanah Khusus	Perlu evaluasi khusus di tiap lokasi	Perlu evaluasi khusus di tiap lokasi	Perlu evaluasi khusus di tiap lokasi

Catatan : semua jenis tanah lempung lunak dengan tebal total lebih dari 3 m dengan PI > 20, $w_n \geq 40$ dan $S_u < 25$ kPa dinyatakan sebagai tanah lunak

JENIS TANAH & PERCEPATAN PUNCAK MUKA TANAH

Tabel Percepatan puncak batuan dasar dan percepatan puncak muka tanah,

Wilayah gempa	Percepatan puncak batuan dasar ('g')	Percepatan puncak muka tanah A_0 ('g')			
		Tanah keras	Tanah sedang	Tanah lunak	Tanah khusus
1	0,03	0,04	0,05	0,08	Diperlukan
2	0,10	0,12	0,15	0,20	Evaluasi
3	0,15	0,18	0,23	0,30	khusus di
4	0,20	0,24	0,28	0,34	setiap lokasi
5	0,25	0,28	0,32	0,36	
6	0,30	0,33	0,36	0,38	

JENIS TANAH & PERCEPATAN PUNCAK MUKA TANAH

Faktor Keutamaan I adalah suatu faktor yang dikalikan pada pengaruh Gempa Rencana untuk menyesuaikan periode ulangnya dengan Kategori bangunan yang ditinjau.

Tabel Faktor Keutamaan (I) untuk berbagai kategori gedung dan bangunan

Kategori gedung	Faktor Keutamaan		
	I_1	I_2	I
Gedung umum seperti untuk penghunian, perniagaan dan perkantoran	1,0	1,0	1,0
Monumen dan bangunan monumental	1,0	1,6	1,6
Gedung penting pasca gempa seperti rumah sakit, instalasi air bersih, pembangkit tenaga listrik, pusat penyelamatan dalam keadaan darurat, fasilitas radio dan televisi.	1,4	1,0	1,4
Gedung untuk menyimpan bahan berbahaya seperti gas, produk minyak bumi, asam, bahan beracun.	1,6	1,0	1,6
Cerobong, tangki di atas menara	1,5	1,0	1,5

Catatan :
Untuk semua struktur bangunan gedung yang ijin penggunaannya diterbitkan sebelum berlakunya Standar ini maka Faktor Keutamaan, I, dapat dikalikan 80%.

Pengaruh Gempa Vertikal

- Pengaruh gempa vertikal khususnya harus ditinjau pada balkon, kanopi, kantilever panjang, balok transfer, balok beton pratekan berbentang panjang, bersamaan dengan pengaruh beban gempa horisontal.

  Dalam arah vertikal bangunan dapat dianggap sepenuhnya mengikuti pergerakan vertikal muka tanah, sehingga faktor respons gempa vertikal :

$$\boxed{C_v = \psi . A_o . I}$$

Dengan nilai ψ tergantung pada wilayah gempa sebagai berikut :

Wilayah Gempa	ψ
1	0,5
2	0,5
3	0,5
4	0,6
5	0,7
6	0,8

SPEKTRUM RESPONS GEMPA RENCANA dan ANALISIS RAGAM SPEKTRUM RESPONS

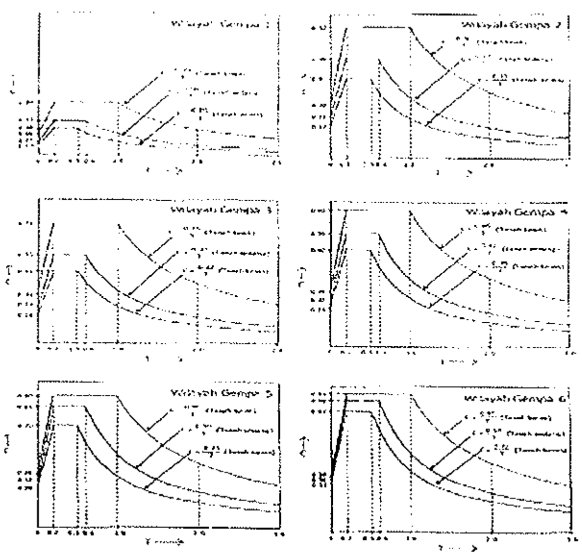
- Spektrum respons Gempa Rencana C-T adalah suatu diagram yang memberi hubungan antara percepatan respons dinamik C (dalam 'g') dan waktu getar alami T (dalam detik) sistim satu derajat kebebasan dengan fraksi redaman kritis 5%.

- Analisis Ragam Spektrum Respons :

Respons dinamik total struktur elastik merupakan superposisi dari respons dinamik ragam-ragamnya, masing-masing ditentukan dari spektrum respons gempa rencana C-T, dikalikan dengan faktor partisipasinya. Dalam hal ini, respons ragam fundamental adalah yang paling dominan partisipasinya.\

Modal Combinations : Cara superposisi respons masing-masing ragam yang paling tepat adalah dengan metode Complete Quadratic Combinations (CQC).

SPEKTRUM RESPONS GEMPA RENCANA dan ANALISIS RAGAM SPEKTRUM RESPONS

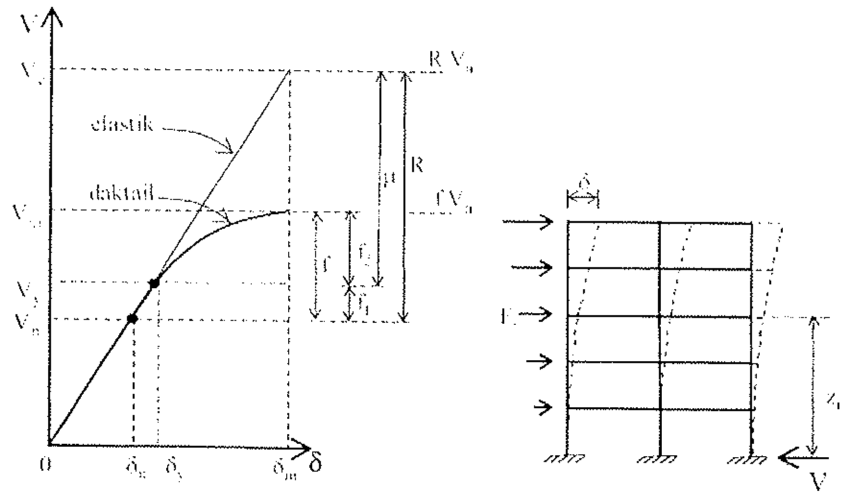


SPEKTRUM RESPONS GEMPA RENCANA dan ANALISIS RAGAM SPEKTRUM RESPONS

Tabel Spektrum respons gempa rencana

Wilayah Gempa	Tanah Keras T_c = 0,5 det.		Tanah Sedang T_c = 0,6 det.		Tanah Lunak T_c = 1,0 det.	
	A_m	A_r	A_m	A_r	A_m	A_r
1	0,10	0,05	0,13	0,08	0,20	0,20
2	0,30	0,15	0,38	0,23	0,50	0,50
3	0,45	0,23	0,55	0,33	0,75	0,75
4	0,60	0,30	0,70	0,42	0,85	0,85
5	0,70	0,35	0,83	0,50	0,90	0,90
6	0,83	0,42	0,90	0,54	0,95	0,95

DAKTILITAS, KUAT LEBIH & PENGARUH GEMPA PADA STRUKTUR GEDUNG



Gambar P.1 Diagram beban-simpangan (diagram V-δ) struktur gedung

DAKTILITAS, KUAT LEBIH & PENGARUH GEMPA PADA STRUKTUR GEDUNG

Diagram *V-δ* didasarkan atas konsep kesamaan simpangan maksimum δ_m pada saat di ambang keruntuhan ; titik pelelehan pertama tergantung pada faktor daktilitas μ :

$$1{,}0 \leq \mu = \frac{\delta_m}{\delta_y} \leq \mu_m$$

δ_y : adalah simpangan akibat beban V_y pada saat pelelehan pertama
$\mu = 1{,}0$: struktur dalam keadaan elastik penuh ($\delta_y = \delta_m$)
$\mu = \mu_m$: struktur pada batas daktilitas maksimumnya
$\mu_m = 5{,}2$ untuk portal terbuka daktail penuh
$\mu_m = 3{,}3$ untuk dinding geser daktail parsial

μ dapat dipilih sendiri oleh perencana atau pemilik gedung, asal tidak melebihi μ_m.

Faktor kuat lebih yang berhubungan dengan faktor keamanan terhadap beban lebih atau beban kurang :

$$f_1 = 1{,}60$$

Faktor kuat lebih yang berhubungan dengan kehiperstatisan struktur daktail sebelum runtuh :

$$f_2 = 0{,}83 + 0{,}17 . \mu$$

DAKTILITAS, KUAT LEBIH & PENGARUH GEMPA PADA STRUKTUR GEDUNG

* Faktor reduksi gempa untuk mendapatkan beban gempa nominal pada struktur daktail :

$$R = \mu.f_1 = 1,6.\mu$$

* Faktor kuat lebih total untuk mendapatkan beban gempa maksimum struktur daktail sebelum runtuh :

$$f = f_1.f_2 = 1,6.f_2$$

* Beban gempa bumi untuk mendesain struktur daktail :

$$V_n = \frac{V_y}{f_1} = \frac{V_e}{R}$$

* Beban gempa maksimum yang dapat dipikul struktur daktail sebelum runtuh :

$$V_m = f.V_n = f_1.f_2.V_n$$

* Beban gempa maksimum nominal untuk mendesain bagian struktur daktail yang harus tetap berperilaku elastik, sekaligus beban gempa nominal terbesar yang dapat bekerja pada struktur daktail :

$$V_{mn} = \frac{V_m}{R} = \frac{f_1.f_2.V_n}{f_1} = f_2.V_n$$

* Bila simpangan akibat beban gempa nominal V adalah δ_n , maka :
  - Simpangan pada saat pelelehan pertama : $\delta_y = f_1.\delta_n$
  - Simpangan pada saat di ambang keruntuhan : $\delta_m = R.\delta_n$

DAKTILITAS, KUAT LEBIH & PENGARUH GEMPA PADA STRUKTUR GEDUNG

Tabel Faktor kuat lebih struktur f2 dan faktor kuat lebih total f yang terkandung di dalam struktur gedung

Taraf kinerja struktur	μ	R pers.(6)	f2 pers.(37)	f pers.(39)
Elastik penuh	1,0	1,6	1,00	1,6
Daktail parsial	1,5	2,4	1,09	1,7
	2,0	3,2	1,17	1,9
	2,5	4,0	1,26	2,0
	3,0	4,8	1,35	2,2
	3,5	5,6	1,44	2,3
	4,0	6,4	1,51	2,4
	4,5	7,2	1,61	2,6
	5,0	8,0	1,70	2,7
Daktail penuh	5,3	8,5	1,75	2,8

DAKTILITAS, KUAT LEBIH & PENGARUH GEMPA PADA STRUKTUR GEDUNG

Pemakaian Gempa Nominal sebesar $f_2.V_n$ pada struktur daktail :

1.Perencanaan Struktur Bawah, yaitu struktur besmen dan pondasi
Akibatnya diharapkan Struktur Bawah akan berperilaku elastik pada saat Struktur Atas berada diambang keruntuhan.

2. Capacity Design
Jika pada pertemuan balok kolom jumlah kapasitas kolom tidak mencapai (6/5) jumlah kapasitas balok, sehingga tulangan kolom harus ditambah, tetapi tidak perlu lebih dari pada untuk memikul gaya dalam $f_2.V_n$, dimana V_n adalah gaya dalam akibat gempa nominal sebelumnya.

3. Perencanaan sistim struktur yang berupa kombinasi portal terbuka dan dinding geser (Sistim Ganda), kalau beban geser yang diterima oleh komponen portal terbuka tidak mencapai 25% beban geser total, sehingga beban geser yang dikerjakan pada komponen portal terbuka harus ditambah, tetapi tidak perlu lebih dari $f_2.V_n$, dimana V_n adalah gaya dalam akibat gempa nominal sebelumnya.

ANALISIS STRUKUR GEDUNG 3D

KETENTUAN UMUM

* Struktur utama penahan beban gempa harus dimodelkan 3 dimensi

* Jika tidak ditinjau iteraksi tanah-struktur, taraf jepitan lateral struktur atas dapat dianggap terjepit pada taraf lantai dasar (kalau ada besmen) dan pada taraf bidang atas pur atau pada taraf bidang telapak (kalau tidak ada besmen).

* Arah gempa yang menentukan : searah dengan bidang kerja subsistim struktur penahan beban gempa yang dominan

* Karena arah gempa pada kenyataannya sembarang, hal ini disimulasikan dengan meninjau beban gempa 100% dalam satu arah, bersamaan dengan beban gempa 30% dalam arah tegak lurusnya.

ANALISIS STRUKUR GEDUNG 3D

KETENTUAN UMUM

Analisis struktur dilakukan dengan memperhitungkan pengaruh retak pada penampang Beton :

Modulus elastisitas		Ec (dari 10.5(1))
Momen Inersia		0,35 *Ig*
Balok		0,70 *Ig*
Kolom		0,70 *Ig*
Dinding	: tidak retak	0,35 *Ig*
	: retak	
Pelat datar dan lantai datar (flat plate/flat slab)		0,25 *Ig*
Luas		1,0 *Ag*

ANALISIS STRUKUR GEDUNG 3D

KETENTUAN UMUM

* Faktor reduksi gempa dihitung sebagai nilai rata - rata berbobot dari R semua unsur vertikal dengan gaya geser dasar V sebagai faktor pembobotnya :

- arah sumbu - x : $R_s = \dfrac{\sum V_{xs}}{\sum (V_{xs}/R_{xs})} = \dfrac{V_x^o}{\sum (V_{xs}/R_{xs})}$

- arah sumbu - y : $R_s = \dfrac{\sum V_{ys}}{\sum (V_{ys}/R_{ys})} = \dfrac{V_y^o}{\sum (V_{ys}/R_{ys})}$

- representatif secara keseluruhan :

$$R = \frac{V_x^o + V_y^o}{\dfrac{V_x^o}{R_x} + \dfrac{V_y^o}{R_y}}$$

* Nilai R masing - masing unsur vertikal tergantung pada nilai yang dipilih, tetapi tidak boleh diambil lebih dari nilai maksimum R_m

  R_m = 8,5 untuk portal terbuka daktail penuh

  R_m = 5,5 untuk dinding geser daktail parsial/biasa

ANALISIS STRUKUR GEDUNG 3D

KETENTUAN UMUM

- Untuk struktur gedung dengan tinggi > 10 tingkat atau 40 m, pengaruh P-Delta harus diperhitungkan.
- Eksentrisitas rencana antara Pusat Massa dan Pusat Rotasi di setiap lantai tingkat harus diperhitungkan untuk mensimulasikan :
  - pengaruh komponen rotasi horisontal gerakan tanah
  - kemungkinan perpindahan letak Pusat Massa karena perubahan dalam beban gravitasi/hidup
  - kemungkinan perpindahan letak Pusat Rotasi karena pengaruh plastifikasi pasca elastik
- Waktu getar alami fundamental : $T1 < \psi . n$.

  n adalah jumlah tingkat dan ψ koefisien menurut tabel berikut :

Wilayah Gempa	ψ
1	0,5
2	0,5
3	0,5
4	0,6
5	0,7
6	0,8

ANALISIS STRUKUR GEDUNG 3D

SNI 03-1726-2002

Gedung beraturan → Harus memenuhi 9 ketentuan → -Tidak perlu Analisis Dinamik
- Statik Ekwivalen

Gedung tidak beraturan → tidak memenuhi 9 ketentuan → Harus Analisis Dinamik

ANALISIS STRUKUR GEDUNG 3D

STRUKTUR ATAS GEDUNG 3D BERATURAN

Karakteristik dinamik struktur 3D :

Gerak ragam getar alami pertama dominan dalam translasi searah dengan salah satu sumbu utamanya;

Gerak alami kedua dominan dalam translasi searah dengan sumbu utama lainnya yang orthogonal dengan sumbu utama sebelumnya. Dengan kata lain, struktur 3D beraturan berperilaku seperti struktur 2D dalam arah masing-masing sumbu utamanya. Karena itu respons total struktur sangat dominan ditentukan oleh respons dari ragam fundamentalnya.

Dengan hanya meninjau respons dari ragam fundamentalnya saja dan bentuk ragamnya didekati sebagai garis lurus, maka analisis ragam spektrum respons (CQC) menghasilkan 2 rumus sederhana untuk beban gempa nominal statik ekwivalen :

$$V = \frac{C_1.I}{R}.W_t \qquad F_i = \frac{W_i.z_i}{\sum_{i=1}^{n}(W_i.z_i)}$$

V = gaya geser dasar nominal

F_i = beban gempa pada taraf lantai - i

C_1 = nilai faktor respons gempa dari Spektrum Respons Gempa Rencana untuk $T = T_1$

I = faktor keutamaan

R = faktor reduksi gempa representatif untuk μ yang dipilih

W_t = berat total gedung (+ beban hidup yg sesuai)

W_i = berat lantai ke - i (+ beban hidup yg sesuai)

z_i = ketinggian lantai ke - i dari taraf penjepitan lateral

ANALISIS STRUKUR GEDUNG 3D

STRUKTUR ATAS GEDUNG 3 D BERATURAN

Karena dalam masing-masing sumbu utama prilaku struktur mirip dengan struktur 2D, maka untuk masing-masing arah tersebut, waktu getar fundamentalnya bisa didekati dengan Rumus Rayleigh :

$$T_1 = 2\pi\sqrt{\frac{\sum_{i=1}^{n}\left(W_i.d_i^2\right)}{g.\sum_{i=1}^{n}(F_i.d_i)}}$$

F_i = beban gempa statik ekwivalen yg bekerja pada struktur atas di taraf masing-masing lantai ke-I

d_i = simpangan/peralihan horisontal struktur di taraf masing - masing lantai ke - i

W_t = berat masing–masing lantai ke - i (beban hidup yang sesuai)

ANALISIS STRUKUR GEDUNG 3D

STRUKTUR ATAS GEDUNG 3 D TIDAK BERATURAN

- Harus dilakukan analisis dinamik untuk menentukannya :
  - waktu-waktu getar alami dan bentuk ragamnya
  - respons dinamik terhadap Gempa Rencana sebagai dasar untuk mendesain struktur
- Untuk mencegah struktur terlalu fleksibel, waktu getar fundamental dibatasi
- Agar struktur berkinerja baik terhadap gempa, gerak ragam getar alami fundamental (kalau bisa juga gerak ragam getar alami kedua) harus dominan dalam translasi, mengingat respons dinamik total struktur terhadap gempa akan dominan ditentukan oleh respons ragam fundamentalnya.
- Bila gerak ragam getar alami fundamental dominan dalam rotasi horisontal (puntir, torsi), gerak respons dinamik total struktur terhadap gempa jug akan dominan dalam rotasi. Hal ini harus dicegah mengingat respons struktur yang berotasi akan menyebabkan :
  - Tuntutan terhadap struktur untuk mengerahkan kekuatan yang berlebihan.
  - Gangguan terhadap kenyamanan penghuni, karena akan merasa pusing, mual dan mengalami disorientasi.
  - Kerusakan non-struktural yang lebih banyak.

ANALISIS STRUKUR GEDUNG 3D

STRUKTUR ATAS GEDUNG 3 D TIDAK BERATURAN

Gaya geser dasar nominal sebagai respons dari ragam getar fundamental (dengan T) :

$$V_1 = \frac{C_1 . I}{R} . W_t$$

C_1 = nilai faktor respons gempa dari Spektrum Respons Gempa Rencana untuk T = T1
I = faktor keutaman gedung berdasarkan fungsi gedung ybs
R = faktor reduksi gempa representatif untuk μ yang dipilih
W_t = berat total gedung (+ beban hidup yg sesuai)

Gaya geser dasar nominal total VCQC = Vt dari hasil analisis ragam spektrum respons untuk mendesain struktur atas harus memenuhi :

$$V_t = V_{CQC} \geq 0{,}8\ V_1$$

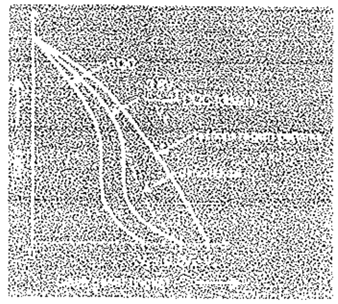
Bila syarat ini tidak terpenuhi, gaya geser tingkat nominal sepanjang tinggi struktur gedung untuk desain harus dikaliakan dengan Faktor Skala :

$$\text{Faktor Skala} = \frac{0{,}8\ V_1}{V_t} \geq 1{,}0$$

Diagram gaya geser nominal sepanjang tingi gedung untuk desain dapat dimodifikasi bila dianggap perlu

ANALISIS STRUKUR GEDUNG 3D

STRUKTUR ATAS GEDUNG 3 D TIDAK BERATURAN



Gambar P.4 Diagram gaya geser tingkat nominal sepanjang tinggi struktur gedung.

Dari diagram gaya geser tingkat nominal sepanjang tinggi struktur gedung didapat beban gempa nominal statik ekwivalen pada taraf masing-masing lantai tingkat, sebagai selisih gaya geser tingkat di bawah dan diatasnya.

Analisis Struktur dari Struktur Bawah

- Struktur Bawah : Besmen dan Pondasi
- Struktur Bawah dapat dianggap sebagai struktur 3D tersendiri di dalam tanah yang mengalami pembebanan gempa yang berasal dari : struktur atas, gaya inersia sendiri dari tanah sekelilingnya.
- Struktur bawah tidak boleh gagal lebih dulu dari struktur atas, karena itu harus selalu berprilaku elastis penuh, sehingga beban gempa maksimum nominal dari struktur ats pada struktur bawah adalah :

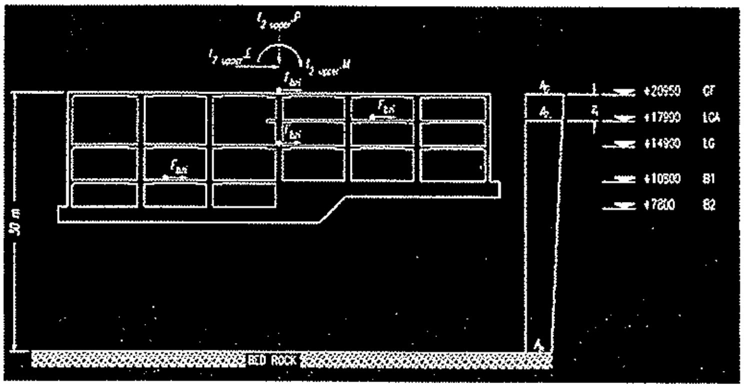
$$V_{mn} = f_{2\,upper} \cdot V_{n\,upper}$$

Beban gempa statik ekwivalen nominal yang bekerja horisontal pada taraf lantai besmen akibat gaya inersia (empirik) :

$$F_{bn} = 0{,}10.A_o.I.W_b$$

A_o = percepatan puncak pada permukaan tanah
I = faktor keutaman gedung
W_b = berat lantai besmen (+ beban hidup yg sesuai)

Analisis dan Desain Struktur dari Struktur Bawah



- Akibat beban horizontal gempa statik ekuivalen (*Fbzi*) :
- *Fbzi* = 0.10 *Azi* . *Wbzi*

Keterangan Notasi :
P, M, E : Reaksi-reaksi dari struktur atas
f1 : 1.6
μD : *R upper* / *f1*
f2 upper : 0.83 + 0.17. *μD*
A0 : Percepatan puncak muka tanah
Ab : Percepatan puncak batuan dasar
zi : Kedalaman yang ditinjau dari muka tanah (m)

$$A_{zi} = \left(A_0 - \frac{z_i}{30m}.(A_0 - A_b)\right)$$

Analisis Struktur dari Struktur Bawah

- **Analisis dan Desain Dinding Besmen terhadap Beban Muka :**

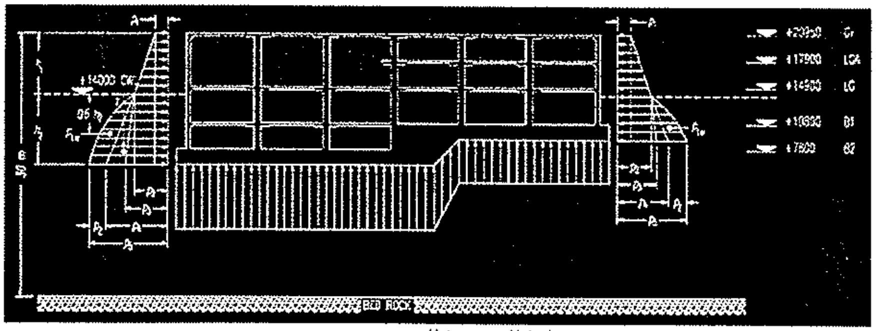
Beban gempa horisontal dari tekanan tanah, dapat dianggap mencapai nilai maksimum senilai tekanan leleh tanah (senilai tekanan pasif tanah) sepanjang kedalaman struktur bawah; beban tanah nominal yang bersangkutan didapat dengan membaginya dengan angka R=1,6; sehingga kombinasi pembebanan terfaktor menjadi :

1,2 DL + 1,6 LL + 1,6 (H/1,6).

Beban gempa horisontal dari tekanan air tanah, dapat dihitung dari massa air yang dibatasi oleh parabola Westergaard, dikalikan dengan percepatan puncak muka tanah Ao; beban hidrodinamik nominal yang bersangkutan didapat dengan membaginya dengan angka R=1,6; sehingga kombinasi pembebanan terfaktor menjadi :

1,2 DL +1,6 LL+ H + F

Analisis dan Desain Struktur Dinding Besmen thd beban muka dari Struktur Bawah



- Akibat beban hidup pada muka tanah (*L*) : *p1* = *q* . *Kp*
- Akibat tekanan tanah (*H*) :
  - *p2* = *p1* + (γt . *h1* . *Kp*)
  - *p3* = *p2* + (γt' . *h2* . *Kp*)
  - *p4* = *p3* + (γw . *h2*)
- Akibat tekanan tanah (*H*) :
  - *p2* = *p1* + (γt . *h1* . *Kp*)
  - *p3* = *p2* + (γt' . *h2* . *Kp*)
  - *p4* = *p3* + (γw . *h2*)

$$p_5 = p_4 + \frac{7}{8}.h_2.\gamma_w\left(A_0 - \frac{h_2}{30m}.(A_0 - A_b)\right)$$

$$p_z = \frac{7}{8}.\sqrt{z.h_2}.\gamma_w.\alpha_h$$

$$p_w = \frac{7}{12}.h_2^{\,2}.\gamma_w.\alpha_h$$

Kombinasi Pembebanan :
1.2 *D* + 1.6 *L* + 1.6 (*H* / 1.6)

Keterangan Notasi :
h1 : Tinggi daerah di atas GWL (m)
h2 : Tinggi daerah di bawah GWL (m)
K0 : *Coefficient of at rest earth pressure for compacted backfill* (= 0.7)
Kp : *Passive Coefficient of earth pressure* (= 1.0)
p : *Total horizontal earth pressure* (kN/m2)
z : Kedalaman yang dilinjau dari muka air tanah (m)

$$\alpha_h = \left(A_0 - \frac{z}{30m}.(A_0 - A_b)\right)$$

γ : *Unit weight of soil* (= 16.5 kN/m3)
γw : *Unit weight of water* (= 10 kN/m3)
γ' : 16.5 - 10 = 6.5 kN/m3

SNI 1726-02

Lateral Load Resistance Structural Systems

- **Bearing wall system**
  is a structural system without an essentially complete space frame that provides support for all or most of the gravity loads.
  Resistance to lateral forces is provided by the same bearing walls acting as shear wall
  - Light frames with shear panels
  - Load bearing shear walls
- **Building Frame System**
  is a structural system with an essentially complete space frame that support the gravity loads. Resistance to lateral forces is provided by shear walls. No interaction between the shear walls and frames is considered in the lateral load analysis; all of the lateral forces are allocated to the walls.
  - Shear Walls (SW) and Moment Resisting Frame
  - Diagonal Bracing (DB)
- **Moment Resisting Frames System (MRFS)**
  is a structural system with an essentially complete space frame providing for gravity loads. Lateral forces are resisted primarily by flexural action of the frame members.The entire space frame may be designated as the seismic-force-resisting system.
  - Special Moment-Resisting Frames (SMRFS)
  - Concrete Intermediate Moment-Resisting Frame (IMRFS)
  - Ordinary Moment-Resisting Frame (OMRFS)
- **Dual Systems (DS)**

a. An essentially complete space frame provides support for gravity loads.
b. Resistance to lateral forces is provided by moment resisting frames capable of resisting at least 25 percent of the design base shear and by shear wall
c. The two subsystems (moment-resisting frames and shear walls) are designed to resist the design base shear in proportion to their relative rigidities
- Special Shear Walls + Special Moment Resisting Frame
- Ordinary Braced Frame (OBF)
- Special Braced Frame (SBF)

Tabel 2.3 Faktor daktilitas maksimum, faktor reduksi gempa maksimum, faktor tahanan lebih struktur dan faktor tahanan total beberapa jenis sistem dan subsistem struktur gedung

SNI 03-2847-02

TATA CARA PERHITUNGAN STRUKTUR BETON
UNTUK BANGUNAN GEDUNG
SNI 03-2847-2002 (PASAL 23-Desain Tahan Gempa)

Ketentuan Dasar

a. Kuat Tekan beton struktural minimum = 17,5 MPa (K-210)
b. Kuat Tekan beton struktural minimum untuk
   Struktur Tahan Gempa = 20 MPa (K-250)
c. Harus menggunakan baja ulir dengan maksimum fy=400 MPa, baja
   polos hanya diperkenankan untuk tulangan spiral atau tendon.
d. Batasan di atas tidak berlaku untuk jaring kawat baja polos.

Tabel 2.3 Faktor daktilitas maksimum, faktor reduksi gempa maksimum, faktor tahanan lebih struktur dan faktor tahanan total beberapa jenis sistem dan subsistem struktur gedung

Sistem dan Subsistem Struktur Gedung	Uraian Sistem Pemikul Beban Gempa	μ_m	Rm pers.(10)	f
1. Sistem dinding penumpu (Sistem struktur yang tidak memiliki rangka ruang pemikul beban gravitasi secara lengkap. Dinding penumpu atau sistem bresing memikul hampir semua beban gravitasi. Beban lateral dipikul diding geser atau rangka bresing).	1. Dinding Geser beton bertulang	2,7	4,5	2,8
	2. Dinding penumpu dengan rangka baja ringan dan bresing tarik	1,8	2,8	2,2
	3. Rangka bresing dimana bresingnya memikul beban gravitasi			
	a. Baja	2,8	4,4	2,2
	b. Beton bertulang (tidak untuk wilayah 5 & 6)	1,8	2,8	2,2
2. Sistem rangka gedung (sistem struktur yang pada dasarnya memiliki rangka ruang pemikul beban gravitasi secara lengkap. Beban lateral dipikul dinding geser atau rangka bresing).	1. Rangka bresing eksentris baja (RBE)	4,3	7,0	2,8
	2. Dinding geser beton bertulang	3,3	5,5	2,8
	3. Rangka bresing biasa			
	a. Baja	3,6	5,6	2,2
	b. Beton bertulang (tidak untuk wilayah 5 & 6)	3,6	5,6	2,2
	4. Rangka bresing konsentrik khusus			
	a. Baja	4,1	6,4	2,2
	5. Dinding geser beton bertulang berangkai daktail	4,0	6,5	2,8
	6. Dinding geser beton bertulang kantilever daktail penuh	3,6	6,0	2,8
	7. Dinding geser beton bertulang kantilever daktail parsial	3,3	5,5	2,8
3. Sistem rangka pemikul momen (Sistem struktur yang pada dasarnya memiliki rangka ruang pemikul beban gravitasi secara lengkap. Beban lateral dipikul rangka pemikul momen terutama melalui mekanisme lentur)	1. Rangka pemikul momen khusus (SRPMK)			
	a. Baja	5,2	8,5	2,8
	b. Beton bertulang	5,2	8,5	2,8
	2. Rangka pemikul momen menengah beton (SRPMM)	3,3	5,5	2,8
	3. Rangka pemikul momen biasa (SRPMB)			
	a. Baja	2,7	4,5	2,8
	b. Beton bertulang	2,1	3,5	2,8
	4. Rangka batang baja pemikul momen khusus (SRBPMK)	4,0	6,5	2,8
4. Sistem ganda (terdiri dari : 1) rangka ruang yang memikul seluruh beban gravitasi; 2) pemikul beban lateral berupa dinding geser atau rangka bresing dengan rangkapemikul momen. Rangka pemikul momen harus direncanakan secara terpisah mampu memikul sekurang-kurangnya 25% dari seluruh beban lateral; 3) kedua sistem harus direncanakan untuk memikul secara bersama-sama seluruh beban lateral dengan memperhatikan interaksi / sistem ganda).	1. Dinding Geser			
	a. Beton bertulang dengan SRPMK beton bertulang	5,2	8,5	2,8
	b. Beton bertulang dengan SRPMB baja	2,6	4,2	2,8
	c. Beton bertulang dengan SRPMM beton bertulang	4,0	6,5	2,8
	2. RBE baja			
	a. Dengan SRPMK baja	5,2	8,5	2,8
	b. Dengan SRPMB baja	2,6	4,2	2,8
	3. Rangka bresing biasa			
	a. Baja dengan SRPMK baja	4,0	6,5	2,8
	b. Baja dengan SRPMB baja	2,6	4,2	2,8
	c. Beton bertulang dengan SRPMK beton bertulang (tidak untuk wilayah 5 & 6)	4,0	6,5	2,8
	d. Beton bertulang dengan SRPMM beton bertulang (tidak untuk wilayah 5 & 6)	2,6	4,2	2,8
	4. Rangka bresing konsentrik khusus			
	a. Baja dengan SRPMK baja	4,6	7,5	2,8
	b. Baja dengan SRPMB baja	2,6	4,2	2,8
5. Sistem struktur gedung kolom kantilever (Sistem struktur yang memanfaatkan kolom kantilever untuk memikul beban lateral)	Sistem struktur kolom kantilever	1,4	2,2	2
6. Sistem interaksi dinding geser dengan rangka	Beton bertulang biasa (tidak untuk wilayah 3,4,5 & 6)	3,4	5,5	2,8
7. Subsistem tunggal (Subsistem struktur bidang yang membentuk struktur gedung secara keseluruhan)	1. Rangka terbuka baja	5,2	8,5	2,8
	2. Rangka terbuka beton bertulang	5,2	8,5	2,8
	3. Rangka terbuka beton bertulang dengan balok beton pratekan (bergantung pada indeks baja total)	3,3	5,5	2,8
	4. Dinding geser beton bertulang berangkai daktail penuh	4,0	6,5	2,8
	5. Dinding geser beton bertulang kantilever daktail parsial	3,3	5,5	2,8

SNI 03-2847-02

Kombinasi Pembebanan (SNI 03-2847-2002, pasal 11.2)

- Kombinasi Beban Mati dan Beban Hidup :
  $U = 1{,}4\ D$
  $U = 1{,}2\ D + 1{,}6\ L + 0{,}5\ (L_{roof}$ atau $R)$
  L_{roof} = Beban Hidup Atap dan R = Beban Air Hujan

- Jika Beban Angin diperhitungkan :
  $U = 1{,}2\ D + f1\ L \pm 1{,}6\ W + 0{,}5\ (L_{roof}$ atau $R)$
  $U = 0{,}9\ D \pm 1{,}6\ W$

- Jika Beban Gempa diperhitungkan :
  $U = 1{,}2\ D + f1\ L \pm E$
  $U = 0{,}9\ D \pm E$

SNI 03-2847-02

Faktor Reduksi Kapasitas

1. Lentur, tanpa beban aksial....................................0,80
2. Beban Aksial dan Beban Aksial dengan lentur
   a. Aksial tarik dan aksial tarik dengan lentur..............0,80
   b. Aksial tekan dan aksial tekan dengan lentur
      - komponen struktur dengan tulangan spiral.............0,70
      - komponen struktur lainnya....................................0,65
3. Geser dan Torsi..0,75
4. Tumpuan pada beton..0,65
5. Beton polos struktural..0,55

SNI 03-2847-02

Korelasi Kriteria Desain Kegempaan dalam beberapa Peraturan yang ada

Codes,Standards, or resource documents	Low/Ordinary (ACI 21.2.1.2)/ SNI 23.2.1.2	Moderate / Intermediate (ACI 21.2.1.3) /SNI 23.2.1.3	High/Special (ACI 21.2.1.4) / SNI 23.2.1.4
IBC 2000, 2003;NFPA 5000 (2003);ASCE 07-98, 7-02; NEHRP 1997,2000	SDC A, B	SDC C	SDC D, E, F
BOCA (1993,1996,1999); Standard Building Code (1994,1997,1999); ASCE 7-93, 7-95, NEHRP (1991, 1994)	SPC A, B	SPC C	SPC D, E
Uniform Building Code (1991, 1994, 1997)	Seismic Zone 0,1	Seismic Zone 2A,2B	Seismic Zone 3,4
SNI 1726-2002	**Seismic Zone 1,2**	**Seismic Zone 3,4**	**Seismic Zone 5,6**

Note:
SDC = Seismic Design Category
SPC = Seismic Perpormance category

Ketentuan Kriteria Desain dan faktor R Pasal 23.2 SNI 03-2847-2002

Resiko Gempa	**Jenis Struktur yang dapat digunakan**	**Faktor Modifikasi Respon (R)**
Rendah	**Sistem Rangka Pemikul Momen**	
	-SRPM Biasa (Bab 3 – Bab 20)	3 – 3,5
	-SRPM Menengah (Pasal 23.10)	5 – 5,5
	-SRPM Khusus (Pasal 23.3-23.5)	8 – 8,5
	Sistem Dinding Struktural	
	-SDS Biasa (Bab 3 – Bab 20)	4 - 4,5
	-SDS Khusus (Pasal 23.6)	5,5 – 6,5
Menengah	**Sistem Rangka Pemikul Momen**	
	-SRPM Menengah (Pasal 23.10)	5 – 5,5
	-SRPM Khusus (Pasal 23.3-23.5)	8 – 8,5
	Sistem Dinding Struktural	
	-SDS Biasa (Bab 3 – Bab 20)	4 - 4,5
	-SDS Khusus (Pasal 23.6)	5,5 – 6,5
Tinggi	**Sistem Rangka Pemikul Momen**	
	-SRPM Khusus (Pasal 23.3-23.5)	8 – 8,5
	Sistem Dinding Struktural	
	-SDS Khusus (Pasal 23.6)	5,5 – 6,5

Dinding Structural : Bearing Wall Systems atau Dual Systems

Pasal-pasal bab 23 SNI 03-2847-02 yg harus dipenuhi untuk berbagai sistem struktur dalam berbagai level gempa

Component Resisting Earthquake Effect	Moderate / Intermediate (pasal 23.2.1.3)	High / Special (pasal 23.2.1.4)
Frame members	Pasal 23.10	Pasal 23.2 ; 23.3 ; 23.4 ;23.5
Structural Walls and coupling beams	-None	Pasal 23.2 ; 23.6
Structural Diaphragms and Trusses	-None	Pasal 23.2 ; 23.7
Foundations	-None	Pasal 23.2 ; 23.8
Frame members not proportioned to resist forces induced by earthquake motions	None	Pasal 23.3 ; 23.9
Beton Tanpa Tulangan	Pasal 24.4	Pasal 24.4 ; 24.10.1

Catatan:
Harus memenuhi juga persyaratan-persyaratan Bab 3 – bab 20 SNI 03-2847-2002

SNI 03-2847-02

Sistem Rangka Pemikul Momen :

Sistem Rangka Ruang dengan komponen-komponen struktur dan join-joinnya menahan gaya-gaya yang bekerja melalui aksi lentur, geser dan aksial.

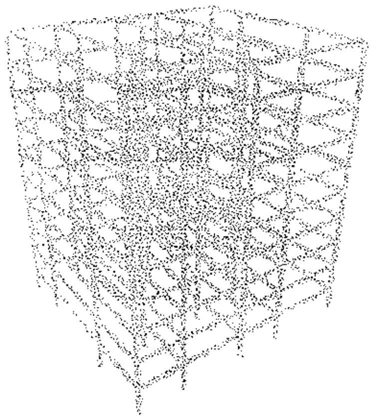
SRPM dikelompokkan sebagai berikut :

a. Sistem Rangka Pemikul Momen Biasa :
Sistem rangka yang memenuhi ketentuan-ketentuan pasal 3 hingga pasal 20

b. Sistem Rangka Pemikul Momen Menengah :
Sistem rangka yang memenuhi ketentuan-ketentuan pasal 3 hingga pasal 20, juga memenuhi ketentuanketentuan pada pasal 23.2(2(3)) dan 23.10

c. Sistem Rangka Pemikul Momen Khusus :
Sistem rangka yang memenuhi ketentuan-ketentuan pasal 3 hingga pasal 20, juga memenuhi ketentuanketentuan pada pasal 23.2 sampai dengan 23.5

Moment Resisting Frame



- The Load is transferred by shear in columns, that produces moment in columns and in beams
- The Beam-Column connection is crucial for the system to work
- The moments and shear from later loads must be added to those from gravity loads

Design Criteria Table

Type of Check/ Design	Ordinary Moment Resisting Frames (non-Seismic)	Intermediate Moment Resisting Frames (Seismic)	Special Moment Resisting Frames (Seismic)
Column Check (interaction)	NLD[a] Combinations	NLD[a] Combinations	NLD[a] Combinations
Column Design (Interaction)	NLD[a] Combinations $1\% \le \rho \le 8\%$	NLD[a] Combinations $1\% \le \rho \le 8\%$	NLD[a] Combinations $\alpha = 1.0$ $1\% \le \rho \le 6\%$
Column Shears	NLD[a] Combinations	Modified NLD[a] Combinations (earthquake loads doubled) Column capacity $\varphi = 1.0$ and $\alpha = 1.0$	NLD[a] Combinations Column shear capacity $\varphi = 1.0$ and $\alpha = 1.25$
Beam Design Flexure	NLD[a] Combinations	NLD[a] Combinations	NLD[a] Combinations $\rho \le 0.025$ $\rho \ge \frac{3\sqrt{f'_c}}{f_y}$, $\rho \ge \frac{200}{f_y}$
Beam Min Moment Override Check	No Requirement	$M^+_{END} \ge \frac{1}{3} M^-_{END}$ $M^+_{SPAN} \ge \frac{1}{5} \max\{M^+, M^-\}_{END}$ $M^-_{SPAN} \ge \frac{1}{5} \max\{M^+, M^-\}_{END}$	$M^+_{END} \ge \frac{1}{2} M^-_{END}$ $M^+_{SPAN} \ge \frac{1}{4} \max\{M^+, M^-\}_{END}$ $M^-_{SPAN} \ge \frac{1}{4} \max\{M^+, M^-\}_{END}$
Beam Design Shear	NLD[a] Combinations	Modified NLD[a] Combinations (earthquake loads doubled) Beam Capacity Shear (V_p) with $\alpha = 1.0$ and $\varphi = 1.0$ plus V_{D+L}	NLD[a] Combinations Beam Capacity Shear (V_p) with $\alpha = 1.25$ and $\varphi = 1.0$ plus V_{D+L} $V_c = 0$
Joint Design	No Requirement	No Requirement	Checked for shear
Beam/Column Capacity Ratio	No Requirement	No Requirement	Reported in output file

NLD[a] = Number of specified loading

SNI 03-2847-02

Sistem Dinding Struktural :

Dinding yang diproporsikan untuk menahan kombinasi geser, momen dan gaya aksial yang ditimbulkan gempa

Ada 3 sistim Dinding Struktural :

- Bearing Wall Systems
- Building Frame Systems (no interaction between Frame & Structural Wall)
- Dual Systems (MRF and Structural Wall Systems)

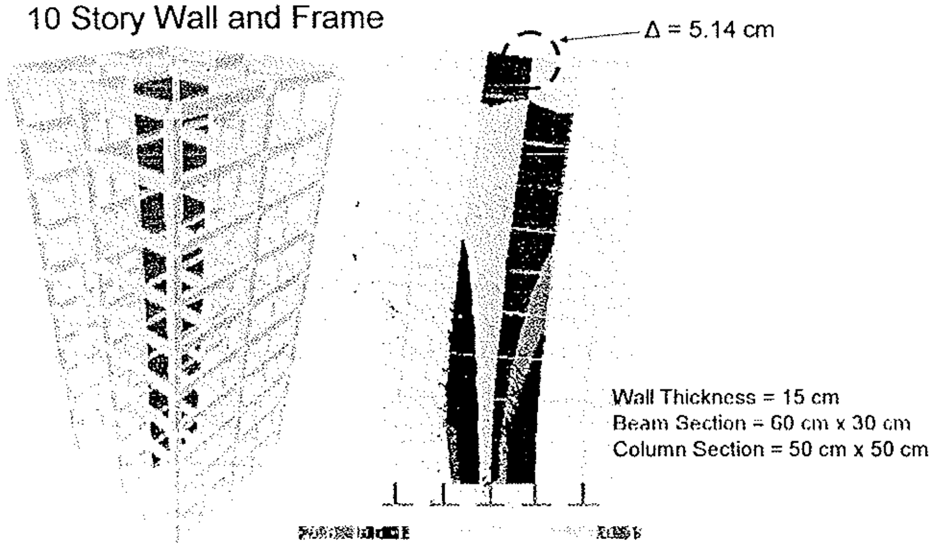
Kriteria Desain Dinding Struktural dikelompokkan sebagai berikut :

a. Dinding Struktural Beton Biasa
Dinding struktural yang memenuhi ketentuan-ketentuan pasal 3 hingga 20.

b. Dinding Struktural Beton Khusus
Dinding struktural yang memenuhi ketentuan-ketentuan pasal 3 hingga 20, juga memenuhi ketentuan-ketentuan SNI 03-2847-02, pasal 23.2 dan 23.6

Shear Wall–Frame Interaction

10 Story Wall and Frame



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Thank you

1. Response Spectra Analysis

Diketahui :

$h_1 := 4.2 \cdot m \quad h_2 := 3.9 \cdot m \quad H := h_1 + h_2 \quad H = 8.1\ m$ **Seismic Zone = 5**

$m_1 := 103492.8611 \cdot kg \quad m_2 := 92719.3512 \cdot kg \quad g := 9.8 \cdot \frac{m}{s^2}$

$w_1 := m_1 \cdot g \quad w_1 = 1014230.039\ N \quad w_2 := m_2 \cdot g \quad w_2 = 908649.642\ N$

$w_t := \sum_{i=1}^{2} w_i \quad w_t = 1922879.681\ N$ **Rumah Sakit:** $I := 1.4$ **SMRF:** $R := 8.5$

A. Pada saat Gempa Rencana Ex :

$MPa := 1 \cdot \frac{N}{mm^2} \quad kPa := 1000 \cdot \frac{N}{m^2}$

a. Kurva Spectra Gempa Rencana

Soil Type Evaluation for Seismic Zone - 5 :

Data parameter tanah sampai kedalaman 30 m :

$t_1 := 7.5 \cdot m \quad N_1 := 5 \quad S_1 := 12 \cdot kPa$

$t_2 := 4 \cdot m \quad N_2 := 25 \quad S_2 := 55 \cdot kPa$

$t_3 := 3.5 \cdot m \quad N_3 := 24 \quad S_3 := 50 \cdot kPa$

$t_4 := 4 \cdot m \quad N_4 := 38 \quad S_4 := 97 \cdot kPa$

$t_5 := 6 \cdot m \quad N_5 := 30 \quad S_5 := 75 \cdot kPa$

$t_6 := 5 \cdot m \quad N_6 := 29 \quad S_6 := 139 \cdot kPa$

$$N := \frac{\sum_{i=1}^{6} t_i}{\sum_{i=1}^{6} \left(\frac{t_i}{N_i}\right)} \quad N = 13.138 \qquad S_u := \frac{\sum_{i=1}^{6} t_i}{\sum_{i=1}^{6} \frac{t_i}{S_i}} \quad S_u = 32.435\ kPa$$

a.1 Soil type based on average SPT value (Standard Penetration Resistance) :

$$\text{Soil\_Type\_based\_on\_SPT} := \begin{cases} \text{"Very Dense Soil/Soft Rock, Sc type"} & \text{if } N > 50.0 \\ \text{"Stiff Soil, Sd type"} & \text{if } 15.0 \le N \le 50.0 \\ \text{"Soft Clay Soil, Se Type"} & \text{otherwise} \end{cases}$$

Soil_Type_based_on_SPT = "Soft Clay Soil, Se Type"

a.2 Soil type based on average Su value (Undrained Shear Strength) :

$$\text{Soil\_Type\_based\_on\_Su} := \begin{cases} \text{"Very Dense Soil/Soft Rock, Sc type"} & \text{if } S_u > 100.0 \cdot kPa \\ \text{"Stiff Soil, Sd type"} & \text{if } 50.0 \cdot kPa \le S_u \le 100.0 \cdot kPa \\ \text{"Soft Clay Soil, Se type"} & \text{otherwise} \end{cases}$$

$$\text{Soil\_Type\_based\_on\_Su} = \text{"Soft Clay Soil, Se type"}$$

Results : $A_o := 0.36 \quad A_m := 2.5 \cdot A_o \quad A_m = 0.9 \quad A_r := 0.90$

UBC 97: $C_a := A_o \quad C_v := A_r$

$$T_c := \left(\frac{A_r}{2.5 \cdot A_o}\right) \cdot s \quad T_c = 1\ s \quad T_o := 0.2 \cdot T_c \quad T_o = 0.2\ s$$

Matriks Kekakuan Lateral dan Matriks Massa: $E_c := 25742960202 \cdot \frac{N}{m^2}$

$$KL := \begin{pmatrix} 50343000 & -25094000 \\ -25094000 & 21308000 \end{pmatrix} \cdot \frac{N}{m} \quad MASS := \begin{pmatrix} m_1 & 0 \\ 0 & m_2 \end{pmatrix} \quad MASS = \begin{pmatrix} 103492.861 & 0 \\ 0 & 92719.351 \end{pmatrix} kg$$

Hasil Analisis Eigen :

Untuk Model Portal Bidang dengan dua derajat kebebasan dari perhitungan dengan CAL-86 :

$$\omega_1 := 8.4624 \cdot \frac{rad}{s} \quad T_1 := \frac{2 \cdot \pi}{\omega_1} \quad T_1 = 0.742\ s$$

$$\omega_2 := 25.39 \cdot \frac{rad}{s} \quad T_2 := \frac{2 \cdot \pi}{\omega_2} \quad T_2 = 0.247\ s$$

$$\phi := \begin{pmatrix} 0.0016333 & -0.0026448 \\ 0.0027942 & 0.0017256 \end{pmatrix}$$

b. <u>Menghitung Koefisien Gempa Rencana,Cm</u>

$$C(T_m) := \begin{cases} \left(1 + \frac{1.5 \cdot T_m}{T_o}\right) \cdot \frac{I}{R} & \text{if } 0 \le T_m < T_o \\ (2.5 \cdot A_o) \cdot \frac{I}{R} & \text{if } T_o \le T_m < T_c \\ \left(\frac{A_r \cdot s}{T_m}\right) \cdot \frac{I}{R} & \text{if } T_c \le T_m \end{cases}$$

$$C(T_1) = 0.148 \quad C(T_2) = 0.148$$

$$S_{a1} := C(T_1) \cdot g \quad S_{a1} = 1.453\ \frac{m}{s^2}$$

$$S_{a2} := C(T_2) \cdot g \quad S_{a2} = 1.453\ \frac{m}{s^2}$$

$$SA := \begin{pmatrix} S_{a1} & 0 \\ 0 & S_{a2} \end{pmatrix} \quad SA = \begin{pmatrix} 1.453 & 0 \\ 0 & 1.453 \end{pmatrix} \frac{m}{s^2}$$

c. Menghitung Gaya Geser Dasar Dinamik akibat Gempa Rencana

Earthquake participation factors :

$$L := \phi^T \cdot MASS \cdot \begin{pmatrix} 1 \\ 1 \end{pmatrix} \quad L = \begin{pmatrix} 428.111 \\ -113.721 \end{pmatrix} kg \quad Ln := \begin{pmatrix} L_{1,1} & 0.0 \cdot kg \\ 0.0 \cdot kg & L_{2,1} \end{pmatrix}$$

Modal Mass :

$$Mn := \phi^T \cdot MASS \cdot \phi \quad Mn = \begin{pmatrix} 1 & -0 \\ -0 & 1 \end{pmatrix} kg \qquad Ln = \begin{pmatrix} 428.111 & 0 \\ 0 & -113.721 \end{pmatrix} kg$$

$$FJ := \frac{MASS}{Mn} \cdot \phi \cdot Ln \cdot SA \qquad FJ = \begin{pmatrix} 105126.787 & 45219.373 \\ 161121.783 & -26431.47 \end{pmatrix} N$$

Effective Weight :

$$W := \frac{Ln^2 \cdot g}{Mn} \qquad W = \begin{pmatrix} 1796144.2 & 2.195 \\ 0.155 & 126736.647 \end{pmatrix} N \qquad w_t = 1922879.681\ N$$

Participating Mass Ratio:

$$PM := \frac{W}{w_t} \qquad PM = \begin{pmatrix} 0.934 & 0 \\ 0 & 0.066 \end{pmatrix}$$

$$V_1 := C(T_1) \cdot W_{1,1} \quad V_2 := C(T_2) \cdot W_{2,2} \quad V_1 = 266251.964\ N \quad V_2 = 18786.844\ N$$

Pakai operasi matriks

$$V_{do} := (1 \;\; 1) \cdot FJ \qquad V_{do} = (266248.57 \;\; 18787.904)\ N$$

Complete Quadratic Combinations : **akibat gempa rencana**

$$\zeta := 0.05 \quad r := \frac{\omega_1}{\omega_2} \quad \rho_{1,2} := \frac{8 \cdot \zeta^2 \cdot (1 + r) \cdot r^{\frac{3}{2}}}{\left(1 - r^2\right)^2 + 4\zeta^2 \cdot r \cdot (1 + r)^2} \quad \rho_{1,1} := 1.0 \quad \rho_{2,2} := 1.0 \quad \rho_{2,1} := \rho_{1,2}$$

$$V_{do} := \sqrt{\sum_{n=1}^{2} \sum_{m=1}^{2} \left(V_{do_{1,n}} \cdot \rho_{n,m} \cdot V_{do_{1,m}} \right)} \qquad V_{do} = 266971.023\ N$$

B. Static Base Shear, Faktor Skala ke Gempa Nominal dan Beban Lateral Ekwivalen

$T_f := T_1$ karena L1 > L2, maka mode 1 adalah mode fundamental, juga periode getarnya

a. Deflection Control:

$T := T_f \quad T = 0.742\ s$

$$V_s := \min\left[\frac{A_m \cdot I}{R} \cdot w_t, \left(\frac{A_r \cdot I}{R \cdot T \cdot s^{-1}} \cdot w_t\right)\right] \qquad V_s = 285038.635\ N$$

$$f_L := \max\left(\frac{0.8 \cdot V_s}{V_{do}}, 1\right) \qquad f_L = 1$$

Lendutan lateral :

$$qL := \phi \cdot \frac{Ln}{Mn} \cdot \begin{pmatrix} \frac{S_{a1}}{\omega_1^2} & 0 \\ 0 & \frac{S_{a2}}{\omega_2^2} \end{pmatrix} \cdot f_L \qquad qL = \begin{pmatrix} 14.185 & 0.678 \\ 24.266 & -0.442 \end{pmatrix} mm$$

Simpangan antara tingkat :

$$\Delta_{1,1} := qL_{1,1} \qquad \Delta_{1,2} := qL_{1,2}$$

$$\Delta_{2,1} := \left(qL_{2,1} - qL_{1,1}\right) \qquad \Delta_{2,2} := qL_{2,2} - qL_{1,2}$$

$$\Delta = \begin{pmatrix} 14.185 & 0.678 \\ 10.082 & -1.12 \end{pmatrix} mm$$

CQC

$$\Lambda := \begin{bmatrix} \sqrt{\sum_{n=1}^{2} \sum_{m=1}^{2} \left(\Delta_{1,n} \cdot \rho_{n,m} \cdot \Delta_{1,m}\right)} \\ \sqrt{\sum_{n=1}^{2} \sum_{m=1}^{2} \left(\Delta_{2,n} \cdot \rho_{n,m} \cdot \Delta_{2,m}\right)} \end{bmatrix}$$

Kontrol kondisi beban kerja :

$h_1 = 4.2\ \mathrm{m} \quad h_2 = 3.9\ \mathrm{m}$

$$\Delta_{all} := \begin{pmatrix} \max\left(\frac{0.03}{R}\cdot h_1, 30\cdot mm\right) \\ \max\left(\frac{0.03}{R}\cdot h_2, 30\cdot mm\right) \end{pmatrix} \qquad \Delta = \begin{pmatrix} 14.203 \\ 10.14 \end{pmatrix} mm \quad \Delta_{all} = \begin{pmatrix} 30 \\ 30 \end{pmatrix} mm$$ **oke**

Kontrol kondisi beban ultimate :

$$\Delta_{ult} := 0.7\cdot R\cdot \Delta \qquad \Delta_{batas} := \begin{pmatrix} 0.02\cdot h_1 \\ 0.02\cdot h_2 \end{pmatrix}$$

$$\Delta_{ult} = \begin{pmatrix} 84.507 \\ 60.335 \end{pmatrix} mm \qquad \Delta_{batas} = \begin{pmatrix} 84 \\ 78 \end{pmatrix} mm$$ **oke juga**

b. Strength Control:

Zone 4 : $\zeta := 0.17$ **jmlh tingkat:** $n := 2 \quad H = 8.1\ \mathrm{m}$

$$T_a := 0.0731\cdot H^{\frac{3}{4}}\cdot m^{-0.75}\cdot s \qquad T_a = 0.351\ s$$ **untuk Moment Resisting Frame**

$$T := \begin{cases} T_f & \text{if } T_f \le 1.2\cdot T_a \\ T_a & \text{otherwise} \end{cases} \qquad T = 0.351\ s \quad T_{sni} := \zeta\cdot n\cdot s \quad T_{sni} = 0.34\ s$$ **anggap cukup**

$$V_1 := \min\left[\frac{A_m\cdot I}{R}\cdot w_t, \left(\frac{A_r\cdot I}{R\cdot T\cdot s^{-1}}\cdot w_t\right)\right] \qquad V_1 = 285038.635\ N \quad 0.8V_s = 228030.908\ N$$

$$f_D := \max\left(\frac{0.8\cdot V_1}{V_{do}}, 1\right) \qquad f_D = 1$$ **hasil dinamik lebih besar dari 0,8.V1, jadi tidak perlu modifikasi**

b.1. Gross Response dari Gempa Nominal:

Gaya lateral ekwivalen

$$FJ := f_D\cdot FJ \quad FJ = \begin{pmatrix} 105126.787 & 45219.373 \\ 161121.783 & -26431.47 \end{pmatrix} N$$

CQC

$$F := \begin{bmatrix} \sqrt{\sum_{n=1}^{2}\sum_{m=1}^{2}\left(FJ_{1,n}\cdot\rho_{n,m}\cdot FJ_{1,m}\right)} \\ \sqrt{\sum_{n=1}^{2}\sum_{m=1}^{2}\left(FJ_{2,n}\cdot\rho_{n,m}\cdot FJ_{2,m}\right)} \end{bmatrix}$$

$$F = \begin{pmatrix} 114573.436 \\ 163191.309 \end{pmatrix} N$$

Gaya geser dasar nominal

$V_d := (1\ \ 1)\cdot FJ\quad V_d = (266248.57\ \ 18787.904)\ N$

CQC

$$V_d := \sqrt{\sum_{n=1}^{2}\sum_{m=1}^{2}\left(V_{d_{1,n}}\cdot\rho_{n,m}\cdot V_{d_{1,m}}\right)}\qquad V_d = 266971.023\ N$$

Momen guling: $h_1 = 4.2\ m\quad h_2 = 3.9\ m\quad H = 8.1\ m$

$Mg := \left(h_1\ \ H\right)\cdot FJ\qquad Mg = (1746618.95\ \ -24173.535)\ Nm$

CQC

$$Mg := \sqrt{\sum_{n=1}^{2}\sum_{m=1}^{2}\left(Mg_{1,n}\cdot\rho_{n,m}\cdot Mg_{1,m}\right)}\qquad Mg = 1746708.329\ Nm$$

Lendutan lateral :

$$qL1 := \phi\cdot\frac{Ln}{Mn}\cdot\begin{pmatrix}\frac{S_{a1}}{\omega_1^{\ 2}} & 0\\ 0 & \frac{S_{a2}}{\omega_2^{\ 2}}\end{pmatrix}\cdot f_D\qquad qL1 = \begin{pmatrix}0.014185 & 0.000678\\ 0.024266 & -0.000442\end{pmatrix}m$$

$$qL := \begin{pmatrix}0.012116 & 0.000579\\ 0.020727 & -0.000378\end{pmatrix}m$$

b.2. Member Response

Dari CAL diperoleh matriks peralihan :

$$qUP := \begin{pmatrix}0.48383\cdot10^{-4} & -0.26941\cdot10^{-5}\\ -0.18071\cdot10^{-2} & 0.2749\cdot10^{-4}\\ 0.42542\cdot10^{-20} & -0.84683\cdot10^{-21}\\ -0.87807\cdot10^{-3} & 0.23172\cdot10^{-4}\\ -0.48383\cdot10^{-4} & 0.26941\cdot10^{-5}\\ -0.18071\cdot10^{-2} & 0.2749\cdot10^{-4}\\ 0.62469\cdot10^{-4} & -0.46207\cdot10^{-5}\\ -0.83884\cdot10^{-3} & 0.12832\cdot10^{-3}\\ 0.29282\cdot10^{-20} & -0.15892\cdot10^{-20}\\ -0.4009\cdot10^{-3} & 0.39945\cdot10^{-4}\\ -0.62469\cdot10^{-4} & 0.46207\cdot10^{-5}\\ -0.83884\cdot10^{-3} & 0.12832\cdot10^{-3}\end{pmatrix}$$

$qTOT := \text{stack}(qUP, qL)$

qTOT =

	1	2
1	0.000048383	-0.0000026941
2	-0.0018071	0.00002749
3	0	0
4	-0.00087807	0.000023172
5	-0.000048383	0.0000026941
6	-0.0018071	0.00002749
7	0.000062469	-0.0000046207
8	-0.00083884	0.00012832
9	0	0
10	-0.0004009	0.000039945
11	-0.000062469	0.0000046207
12	-0.00083884	0.00012832
13	0.012116	0.000579
14	0.020727	-0.000378

Data batang 1 :

$c_1 := \cos(90 \cdot deg) \qquad c_2 := \sin(90 \cdot deg)$

$$D1 := \begin{pmatrix} 0 & 0 \\ 0 & 0 \\ 0 & 0 \\ 0.012115597 & 0.000578908 \\ 0.48383 \cdot 10^{-4} & -0.2694 \cdot 10^{-5} \\ -0.18071 \cdot 10^{-2} & 0.2749 \cdot 10^{-4} \end{pmatrix} \qquad \lambda_b := \begin{pmatrix} c_1 & c_2 & 0 \\ -c_2 & c_1 & 0 \\ 0 & 0 & 1 \end{pmatrix} \qquad \lambda_a := \lambda_b$$

$$D_a := \text{submatrix}(D1, 1, 3, 1, 2) \qquad D_a = \begin{pmatrix} 0 & 0 \\ 0 & 0 \\ 0 & 0 \end{pmatrix}$$

$$D_b := \text{submatrix}(D1, 4, 6, 1, 2) \qquad D_b = \begin{pmatrix} 0.012116 & 0.000579 \\ 0.000048 & -0.000003 \\ -0.001807 & 0.000027 \end{pmatrix}$$

$$d_b := \lambda_b \cdot D_b \qquad d_b = \begin{pmatrix} 0.000048 & -0.000003 \\ -0.012116 & -0.000579 \\ -0.001807 & 0.000027 \end{pmatrix}$$

$$d_a := \lambda_a \cdot D_a \qquad d_a = \begin{pmatrix} 0 & 0 \\ 0 & 0 \\ 0 & 0 \end{pmatrix}$$

$$d1 := \mathrm{stack}(d_a, d_b)$$

$$d1 = \begin{pmatrix} 0 & 0 \\ 0 & 0 \\ 0 & 0 \\ 0.000048383 & -0.000002694 \\ -0.012115597 & -0.000578908 \\ -0.0018071 & 0.00002749 \end{pmatrix}$$

Satuan konsisten, N, m, rad

$E_c := 25742960202 \qquad A := 0.09 \qquad b_w := 0.3 \qquad h := 0.3 \qquad L1 := 4.2$

$In := \frac{1}{12} \cdot b_w \cdot h \quad I = 1.4 \qquad \alpha := \frac{E_c \cdot In}{L1^3} \quad \alpha = 2605984.795 \quad \beta := \frac{A \cdot L1^2}{In} \quad \beta = 211.68$

$$k1 := \begin{pmatrix} \beta & 0 & 0 & -\beta & 0 & 0 \\ 0 & 12 & 6 \cdot L1 & 0 & -12 & 6 \cdot L1 \\ 0 & 6 \cdot L1 & 4 \cdot L1^2 & 0 & -6 \cdot L1 & 2 \cdot L1^2 \\ -\beta & 0 & 0 & \beta & 0 & 0 \\ 0 & -12 & -6 \cdot L1 & 0 & 12 & -6 \cdot L1 \\ 0 & 6 \cdot L1 & 2 \cdot L1^2 & 0 & -6 \cdot L1 & 4 \cdot L1^2 \end{pmatrix} \cdot \alpha$$

Gaya dalam batang 1 dalam koordinat lokal :

$$f1 := k1 \cdot d1$$

$$f1 = \begin{pmatrix} -26689.75 & 1486.104 \\ 260203.006 & 19908.796 \\ 629497.925 & 40544.768 \\ 26689.75 & -1486.104 \\ -260203.006 & -19908.796 \\ 463354.699 & 43072.175 \end{pmatrix}$$

Hasil CQC :

$$fl := \begin{bmatrix} \sqrt{\sum_{n=1}^{2} \sum_{m=1}^{2} \left(fl_{1,n} \cdot \rho_{n,m} \cdot fl_{1,m}\right)} \\ \sqrt{\sum_{n=1}^{2} \sum_{m=1}^{2} \left(fl_{2,n} \cdot \rho_{n,m} \cdot fl_{2,m}\right)} \\ \sqrt{\sum_{n=1}^{2} \sum_{m=1}^{2} \left(fl_{3,n} \cdot \rho_{n,m} \cdot fl_{3,m}\right)} \\ \sqrt{\sum_{n=1}^{2} \sum_{m=1}^{2} \left(fl_{4,n} \cdot \rho_{n,m} \cdot fl_{4,m}\right)} \\ \sqrt{\sum_{n=1}^{2} \sum_{m=1}^{2} \left(fl_{5,n} \cdot \rho_{n,m} \cdot fl_{5,m}\right)} \\ \sqrt{\sum_{n=1}^{2} \sum_{m=1}^{2} \left(fl_{6,n} \cdot \rho_{n,m} \cdot fl_{6,m}\right)} \end{bmatrix}$$

$$fl = \begin{pmatrix} 26726.309 \\ 261027.494 \\ 630932.657 \\ 26726.309 \\ 261027.494 \\ 465490.517 \end{pmatrix}$$

2. SNI 1726-2002 Static Equivalent Method

$w_1 := m_1 \cdot g \quad w_1 = 1014230.039\ N \quad w_2 := m_2 \cdot g \quad w_2 = 908649.642\ N \quad w_t = 1922879.681$ N

$h_1 := h_1 \quad h_2 := h_1 + h_2 \quad h_2 = 8.1\ m \quad B := 3 \cdot 6m \quad B = 18\ m$

$T_f := T_1$ karena L1 > L2, maka mode 1 adalah mode fundamental, juga periode getarnya

$T_a := 0.0731 \cdot H^{\frac{3}{4}} \cdot m^{-0.75} \cdot s \qquad T_a = 0.351\ s$ **untuk MRF** $\quad R = 8.5$

$$T := \begin{cases} T_f & \text{if } T_f \le 1.2 \cdot T_a \\ T_a & \text{otherwise} \end{cases} \qquad T = 0.351\ s \quad T_{sni} := 0.16 \cdot 2 \cdot s \quad T_{sni} = 0.32\ s$$ cukup oke

$$V_s := \min\left[\frac{A_m \cdot I}{R} \cdot w_t, \left(\frac{A_r \cdot I}{R \cdot T \cdot s^{-1}} \cdot w_t\right)\right] \qquad V_s = 285038.635\ N$$

$$F_t := \begin{cases} 0.0 \cdot N & \text{if } \frac{H}{B} \le 3 \\ 0.1 \cdot V_s & \text{otherwise} \end{cases} \qquad F_t = 0\ N$$

$$FL_1 := \frac{w_1 \cdot h_1}{\sum_{i=1}^{2} (w_i \cdot h_i)} \cdot (V_s - F_t) \qquad FL_2 := \frac{w_2 \cdot h_2}{\sum_{i=1}^{2} (w_i \cdot h_i)} \cdot (V_s - F_t) + F_t$$

Gaya lateral statik ekwivalen

Gaya lateral ekwivalen hasil analisis dinamik :

$$FL = \begin{pmatrix} 164971.153 \\ 285038.635 \end{pmatrix} N \qquad F = \begin{pmatrix} 114573.436 \\ 163191.309 \end{pmatrix} N$$

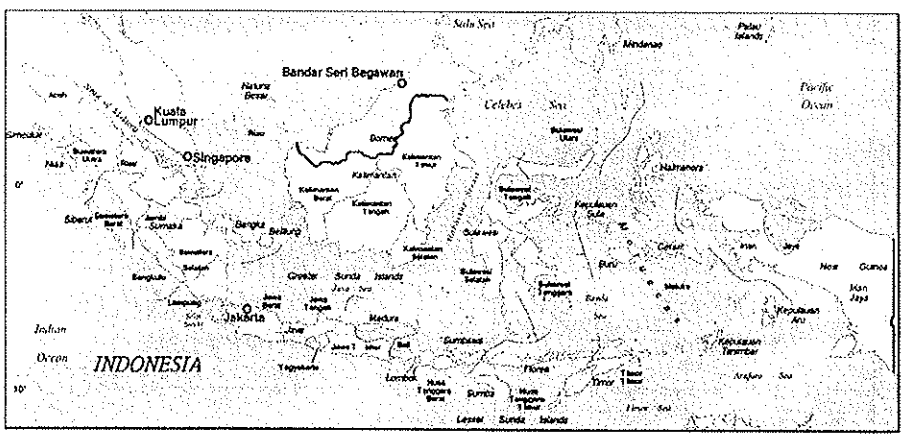
desain akan lebih ekonomis

Seismic Detailing Aspect of Concrete Buildings according to SNI 03-2847-2002

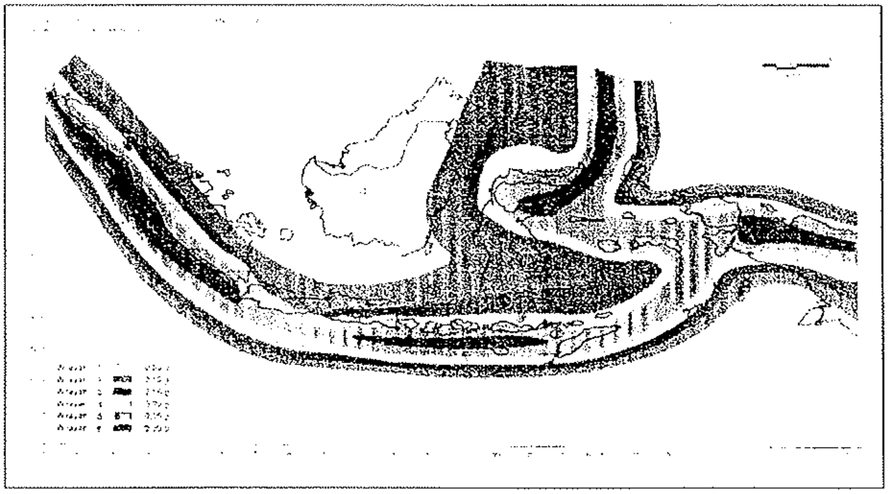
By
Djoni Simanta

Civil Engineering Department
Parahyangan Catholic University
Bandung
2004

Indonesia



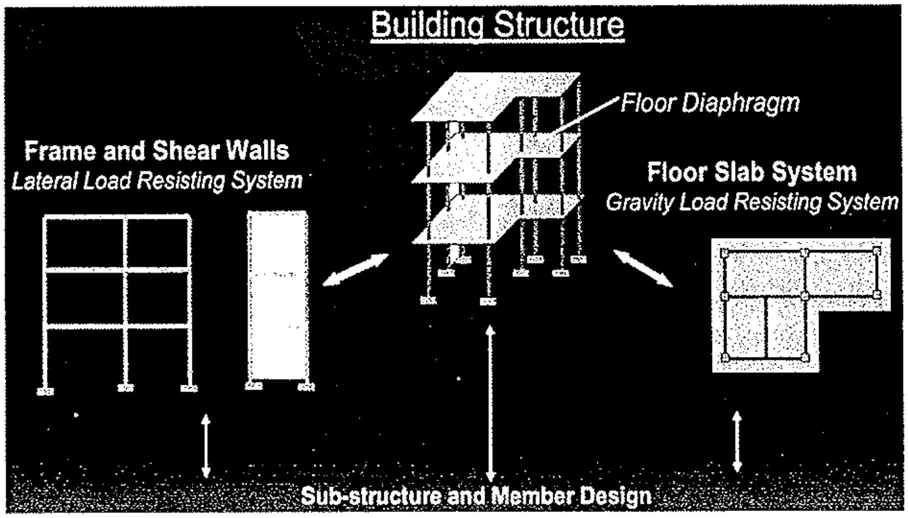
Seismic Zone in Indonesia



NEW

"Tata cara perencanaan struktur beton untuk bangunan gedung SNI 03-2847-2002"

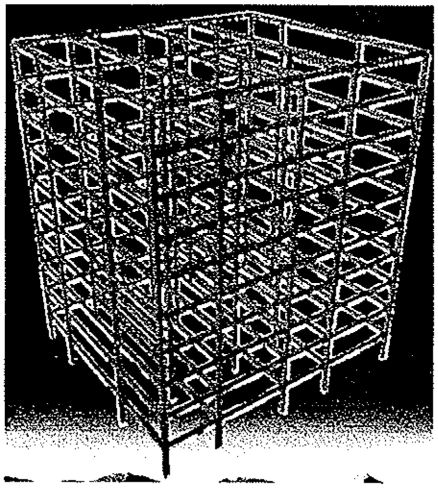
Special provision for Seismic Design of RC Building in Chapter 23 SNI 03-2847-2002 =Chapter 21 ACI 318M-99



Sample Lateral Load Resistance Systems

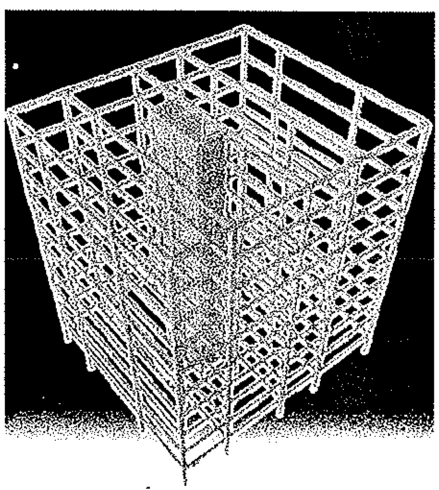
- Bearing wall system
  - Light frames with shear panels
  - Load bearing shear walls
- Fully Braced System (FBS)
  - Shear Walls (SW)
  - Diagonal Bracing (DB)
- Moment Resisting Frames (MRF)
  - Special Moment-Resisting Frames (SMRF)
  - Concrete Intermediate Moment-Resisting Frame (IMRF)
  - Ordinary Moment-Resisting Frame (OMRF)
- Dual Systems (DS)
  - Shear Walls + Frames (SWF)
  - Ordinary Braced Frame (OBF)
  - Special Braced Frame (SBF)

Moment Resisting Frame



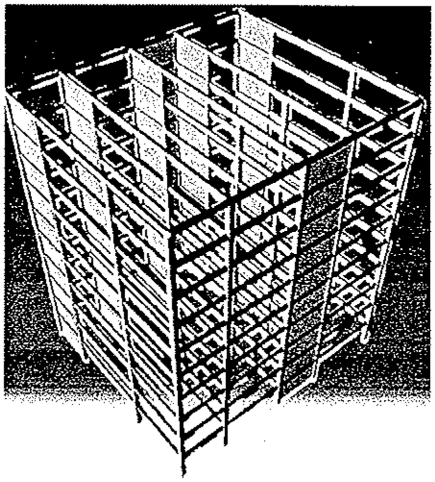
- The Load is transferred by shear in columns, that produces moment in columns and in beams
- The Beam-Column connection is crucial for the system to work
- The moments and shear from later loads must be added to those from gravity loads

Shear Wall and Frame



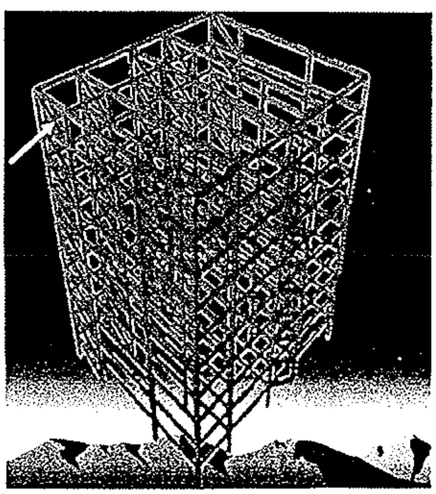
- The lateral loads is primarily resisted by the shear in the walls, in turn producing bending moment
- The openings in wall become areas of high stress concentration and need to be handled carefully
- Partial loads is resisted by the frames
- Traditionally 75/25 distribution haws been used

Shear Wall - Frame



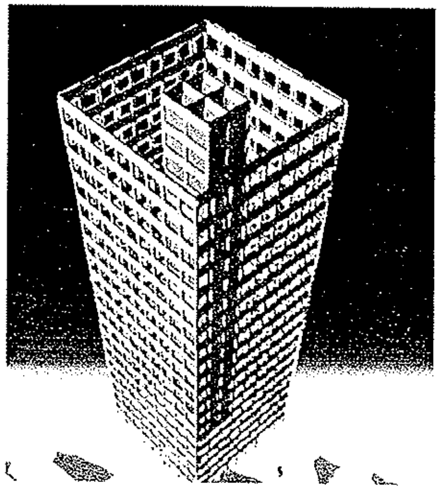
- The Walls are part of the frame and act together with the frame members
- The lateral loads is primarily resisted by the shear in the walls, in turn producing bending moment.
- Partial loads is resisted by the frame members in moment and shear

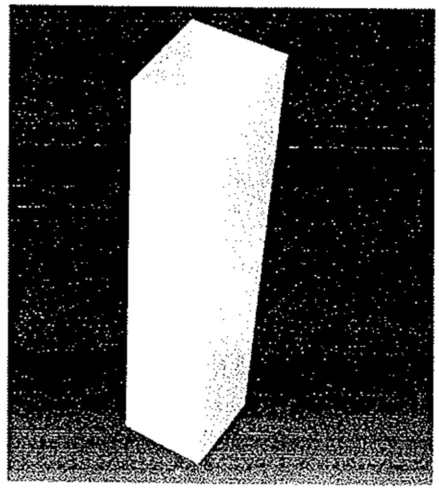
Braced Frame



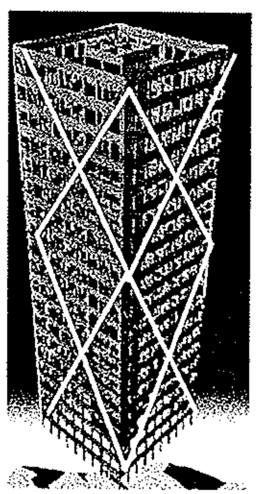
- The lateral loads is primarily resisted by the Axial Force in the braces, columns and beams in the braced zone.
- The frame away from the braced zone does not have significant moments
- Bracing does not have to be provided in every bay, but should be provided in every story

Tubular Structure Tubular Structure





Braced Tube Systems



- Diagonal Braces are added to the basic tubular structure
- This modification of the Tubular System reduces shear lag between opposite faces

General Considerations of Earthquake Resistant Design

The main goal of earthquake-resistant design is to attain a structure with sufficient strength and ductility to assure life safety, i.e., to prevent collapse under the most intense earthquake expected at a site during the life of a structure.

In most structures that are subjected to moderate-to-strong earthquakes, economical earthquake-resistant design is achieved by allowing yielding to take place in some structural members. It is typically impractical as well as uneconomical to design a structure to respond in the elastic range to maximum expected earthquake-induced inertia forces. However, for certain types of structures such as nuclear containment buildings, yielding in the structure cannot be tolerated, and the design needs to be elastic.

In general, most earthquake code provisions implicitly require that structures be able to resist minor earthquakes without any damage, moderate earthquakes with negligible structural damage and some non-structural damage, and major earthquakes with possibly some structural and non-structural damage. As noted above, structures must respond to strong ground motion without collapse.

Designing structures for the effects of earthquakes generally includes the following:

- Selecting and laying out a lateral-force-resisting (LFR) system this is appropriate to the anticipated level of ground shaking. This includes providing a continuous and redundant load path that ensures that a structure acts as an integral unit when responding to ground motion.
- Determining code-prescribed forces and deformations generated by the ground motion, and distributing the forces to the various elements of the LFR system. Site characteristics, occupancy, configuration, structural system, and structure height are all considered when determining these forces.
- Proportioning and detailing the structural members and joints for the combined effects of gravity and lateral (including wind) loads so that adequate vertical and lateral strength and stiffness are achieved to satisfy the structural performance and acceptable deformation levels prescribed in the governing building code.

Table 1. Seismic Risk Terminology

Code,Standard, or resource document	Low/Ordinary (21.2.1.2)/23.2.1.2	Moderate / Intermediate (21.2.1.3) / 23.2.1.3	High/Special (21.2.1.4) / 23.2.1.4
IBC 2000, 2003;NFPA 5000 (2003);ASCE 07-98, 7-02; NEHRP 1997,2000	SDC A, B	SDC C	SDC D, E, F
BOCA (1993,1996,1999); Standard Building Code (1994,1997,1999); ASCE 7-93, 7-95, NEHRP (1991, 1994)	SPC A, B	SPC C	SPC D, E
Uniform Building Code (1991, 1994, 1997)	Seismic Zone 0,1	Seismic Zone 2A,2B	Seismic Zone 3,4
SNI 1726-2002	**Seismic Zone 1,2**	**Seismic Zone 3,4**	**Seismic Zone 5,6**

Note:
SDC = Seismic Design Category
SPC = Seismic Perpormance category

Table 2. Sections of Chapter 23 to Be Satisfied*

Component Resisting Earthquake Effect	Moderate / Intermediate (Chapter 23.2.1.3)	High / Special (Chapter 23.2.1.4)
Frame members	Chapter 23.10	Chapter 23.2 ; 23.3 ; 23.4 ;23.5
Structural Walls and coupling beams	-None	Chapter 23.2 ; 23.6
Structural Diaphragms and Trusses	-None	Chapter 23.2 ; 23.7
Foundations	-None	Chapter 23.2 ; 23.8
Frame members not proportioned to resist forces induced by earthquake motions	-None	Chapter 23.3 ; 23.9
Plain Concrete	Chapter 24.4	Chapter 24.4 ; 24.10.1

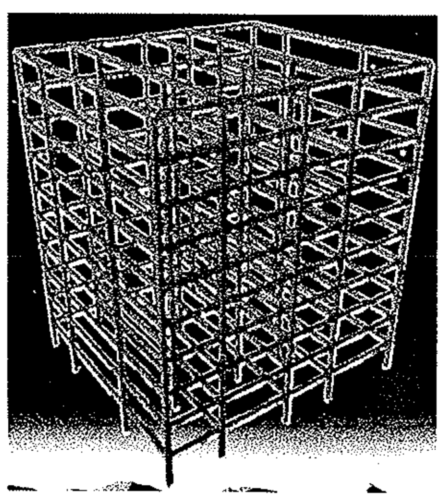
*In addition to requirements of Chapter 1-20

Design Criteria Table

Type of Check/ Design	Ordinary Moment Resisting Frames (non-Seismic)	Intermediate Moment Resisting Frames (Seismic)	Special Moment Resisting Frames (Seismic)
Column Check (interaction)	NLD^a Combinations	NLD^a Combinations	NLD^a Combinations
Column Design (interaction)	NLD^a Combinations $1\% < \rho < 8\%$	NLD^a Combinations $1\% < \rho < 8\%$	NLD^a Combinations $\alpha = 1.0$ $1\% < \rho < 6\%$
Column Shears	NLD^a Combinations	Modified NLD^a Combinations (earthquake loads doubled); Column capacity $\varphi = 1.0$ and $\alpha = 1.0$	NLD^a Combinations Column shear capacity $\varphi = 1.0$ and $\alpha = 1.25$
Beam Design Flexure	NLD^a Combinations	NLD^a Combinations	NLD^a Combinations $\rho \le 0.025$ $\rho \ge \frac{3\sqrt{f'_c}}{f_y}$, $\rho \ge \frac{200}{f_y}$
Beam Min. Moment Override Check	No Requirement	$M^+_{uEND} \ge \frac{1}{3} M^-_{uEND}$ $M^-_{uSPAN} \ge \frac{1}{5} \max\{M^+_u, M^-_u\}_{END}$ $M^+_{uSPAN} \ge \frac{1}{5} \max\{M^+_u, M^-_u\}_{END}$	$M^+_{uEND} \ge \frac{1}{2} M^-_{uEND}$ $M^-_{uSPAN} \ge \frac{1}{4} \max\{M^+_u, M^-_u\}_{END}$ $M^+_{uSPAN} \ge \frac{1}{4} \max\{M^+_u, M^-_u\}_{END}$
Beam Design Shear	NLD^a Combinations	Modified NLD^a Combinations (earthquake loads doubled) Beam Capacity Shear (V_p) with $\alpha = 1.0$ and $\varphi = 1.0$ plus V_{D+L}	NLD^a Combinations Beam Capacity Shear (V_p) with $\alpha = 1.25$ and $\varphi = 1.0$ plus V_{D+L} $V_c = 0$
Joint Design	No Requirement	No Requirement	Checked for shear
Beam/Column Capacity Ratio	No Requirement	No Requirement	Reported in output file

NLD^a = Number of specified loading

Special Moment Resisting Frame



1. Flexural Members of Special Moment Frames

General Requirements

Special moment frames are required in regions of high seismic risk or for structures assigned to high seismic performance or design categories. Flexural members of special moment frames must satisfy the provisions of 23.3. The general requirements of 23.3.1.1 through 23.3.1.4, which are summarized in Fig. 1, have been guided by experimental evidence and observations of reinforced concrete frames that have performed well in the past.

Table 3 - General Requirements

	Sect. No.	Fig. No.
Factored axial compressive force ≤ $A_g f'_c/10$	23.3.1.1	—
Clear span ≥ 4 × effective depth	23.3.1.2	1
Width-to-depth ratio ≥ 0.3	23.3.1.3	
Width ≥ 250 mm.	23.3.1.4	
Width < width of supporting member (measured on a plane perpendicular to the longitudinal axis of the flexural member) + distances on each side of the supporting member not exceeding three-fourths of the depth of the flexural member.		

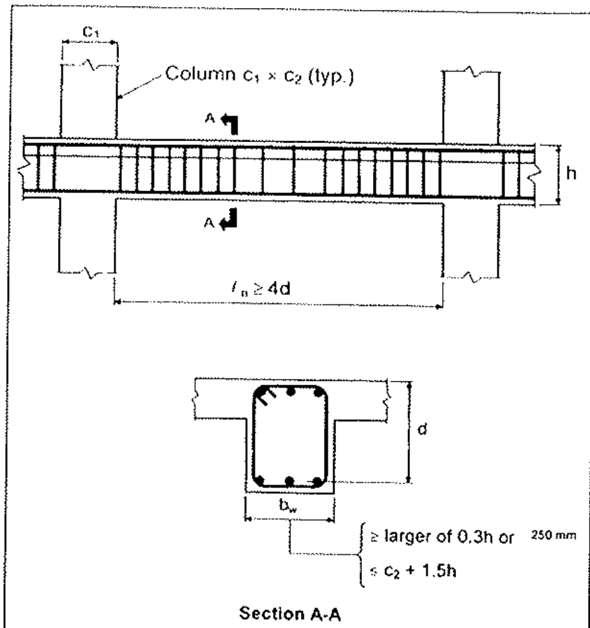


Figure 1 - General Requirements

Table 4 - Flexural Requirements

	Sect. No.	Fig. No.
Minimum reinforcement shall not be less than $\frac{\sqrt{f'_c} b_w d}{4 f_y}$ and $\frac{1.4\, b_w d}{f_y}$ at any section, top and bottom, unless provisions of 10.5.3 are satisfied.	23.3.2.1	2
The reinforcement ratio ρ shall not exceed 0.025.		
At least two bars must be provided continuously at both top and bottom of section.		
Positive moment strength at joint face ≥ 1/2 negative moment strength provided at that face of the joint.	23.3.2.2	
Neither the negative nor the positive moment strength at any section along the member length shall be less than 1/4 the maximum moment strength provided at the face of either joint.		

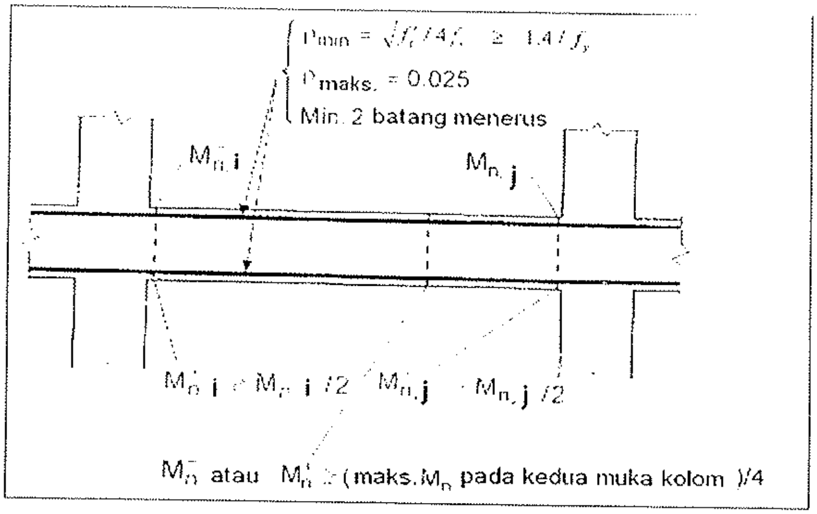


Figure 2 - Flexural Requirements

Table 5 - Splice Requirements

	Sect. No.	Fig. No.
Lap splices of flexural reinforcement are permitted only if hoop or spiral reinforcement is provided over the lap length. Hoop and spiral reinforcement spacing shall not exceed • *d*/4 • 100 mm	23.3.2.3	3
Lap splices are not to be used: • Within joints. • Within a distance of 2h from the face of the joint. • At locations where analysis indicates flexural yielding caused by inelastic lateral displacements of the frame.		
Mechanical splices shall conform to 23.2.6 and welded splices shall conform to 23.2.7.1.	23.3.2.4	—

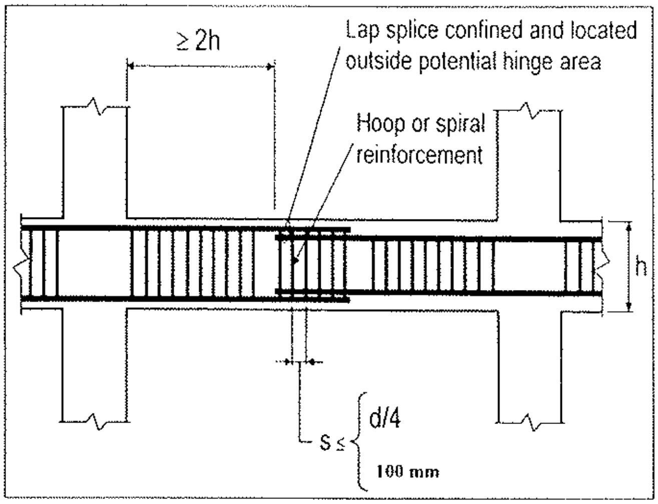


Figure 3 - Lap Splice Requirements

Table 6 - Transverse Reinforcement Requirements

	Sect. No.	Fig. No.
Hoops are required in the following regions of frame members: • Over a length equal to 2h from the face of the supporting member toward midspan at both ends of the flexural member. • Over lengths equal to 2h on both sides of a section where flexural yielding may occur in connection with inelastic lateral displacements of the frame.	23.3.3.1	
Where hoops are required, the spacing shall not exceed: • $d/4$ • 8 × diameter of smallest longitudinal bar • 24 × diameter of hoop bars • 300 mm. The first hoop shall be located no more than 150 mm from the face of the supporting member.	23.3.3.2	4
Where hoops are required, longitudinal bars on the perimeter shall have lateral support conforming to 7.10.5.3.	23.3.3.3	
Where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance not more than $d/2$ throughout the length of the member.	23.3.3.4	
Stirrups or ties required to resist shear shall be hoops over lengths of members in 23.3.3, 23.4.4, and 23.5.2.	23.3.3.5	
Hoops in flexural members shall be permitted to be made up of 2 pieces of reinforcement: a stirrup having seismic hooks at both ends and closed by a crosstie. Consecutive crossties engaging the same longitudinal bar shall have their 90-degree hooks at opposite sides of the flexural member. If the longitudinal bars secured by the crossties are confined by a slab on only one side of the flexural frame member, the 90-degree hooks of the crossties shall be placed on that side.	23.3.3.6	5
Transverse reinforcement must also be proportioned to resist the design shear forces.	23.3.4	4

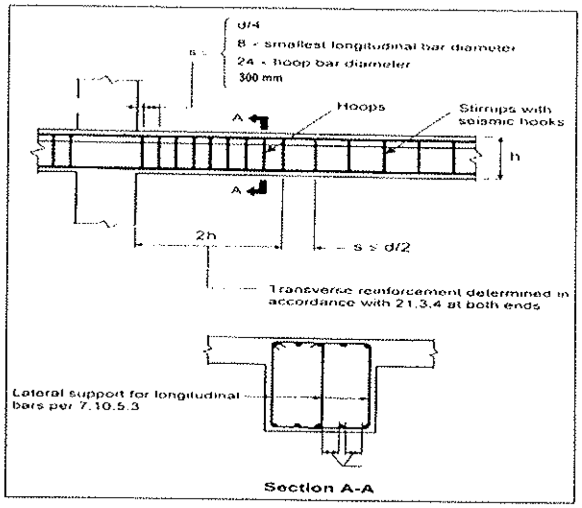


Figure 4 - Transverse Reinforcement Requirements

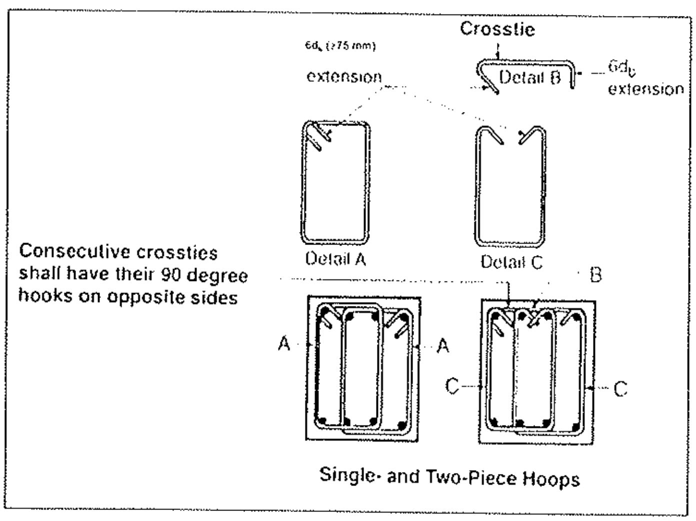


Figure 5 - Hoop Reinforcement

2. Special Moment Frame Members Subjected to Bending and Axial Load

General Requirements

- Special moment frame members subjected to bending and axial load must satisfy the provisions of 23.4. These requirements are for frame members in regions of high seismic risk or for structures assigned to high seismic performance or design categories. The geometric constraints in 23.4.1, which are summarized in Table *7* and Fig. 6, follow from previous practice.

- Note that any frame members in the structure that do not satisfy 23.3.1 are to be proportioned and detailed according to 23.4.

Table 7 - General Requirements

	Sect. No.	Fig. No.
Factored axial compressive force > $A_g f_c'/10$	23.4.1	-
Shortest cross-sectional dimension measured on a straight line passing through the geometric centroid ≥ 300 mm	23.4.1.1	6
Ratio of the shortest cross-sectional dimension to the perpendicular dimension ≥ 0.4	23.4.1.2	

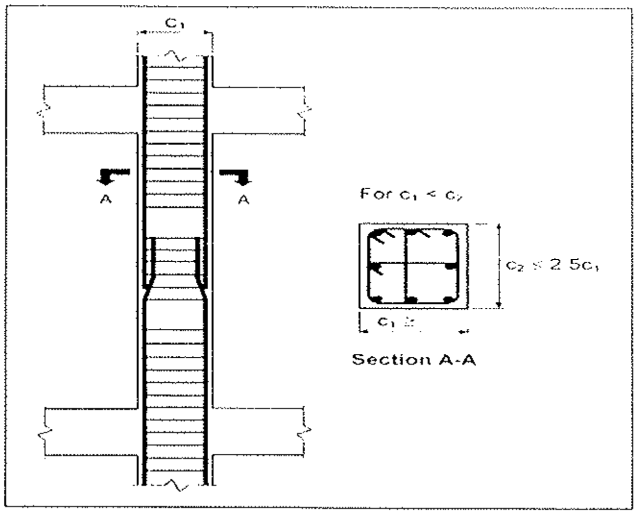


Figure 6 - General Requirements

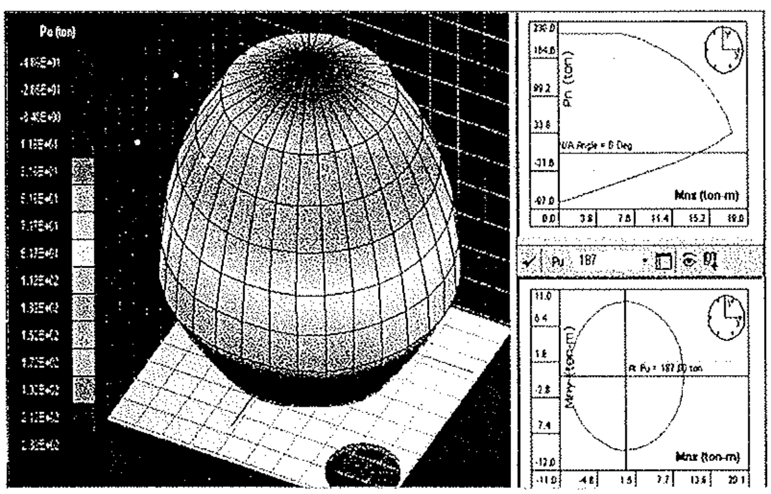
Table 8 - Minimum Flexural Strength of Columns

	Sect. No.	Fig. No.
The flexural strengths of columns shall satisfy the following: $\Sigma\, Mc \geq (6/5)\, \Sigma\, Mg$ (23-1) where $\Sigma\, Mc$ = sum of moments at the faces of the joint, corresponding to the nominal flexural strength of the columns framing into that joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength. $\Sigma\, Mg$ = 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, slab reinforcement within an effective slab width defined in 8.10 shall contribute to flexural strength.	23.4.2.2	-
If Eq. (23-1) is not satisfied, the lateral strength and stiffness of the columns shall not be considered when determining the strength and stiffness of the structure , and the columns shall conform to 23.9. Also, the columns must have transverse reinforcement over their full height as specified in 23.4.4.1 through 23.4.4.3.	23.4.2.1 23.4.2.3	-

Table 9 - Longitudinal Reinforcement Requirements

	Sect. No.	Fig. No.
The reinforcement ratio ρ shall not be less than 0.01 and shall not exceed 0.06	23.4.3.1	
Mechanical splices shall conform to 23.2.6 and welded splices shall conform to 23.2.7. 1. Lap splices are permitted within the center half of the member length, must be only tension lap splices, and shall be enclosed within transverse reinforcement conforming to 23.4.4.2 and 23.4.4.3	23.4.3.2	7

Graphical view of the Capacity



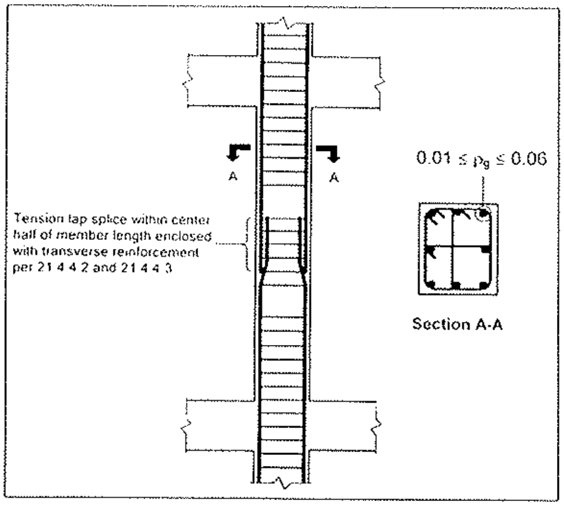


Figure 7 - Longitudinal Reinforcement Requirements

Table 10 - Transverse Reinforcement Requirements (1)

	Sect. No.	Fig. No.
The transverse reinforcement requirements discussed in the following seven items need only be provided over a length ℓ_o from each joint face and on both sides of any section where flexural yielding is likely to occur. The length ℓ_o shall not be less than: • Depth of member at joint face or at section where flexural yielding is likely to occur • Clear span/6 • 500 mm.	23.4.4.4	8
Ratio of spiral or circular hoop reinforcement ρ_s shall not be less than that given by: $\rho_s = 0.12\frac{f'_c}{f_{yh}} \geq 0.45\left(\frac{A_g}{A_c}-1\right)\frac{f'_c}{f_{yh}}$ (23-2) and (10-6)	23.4.4.1(a)	9
Total cross-sectional area of rectangular hoop reinforcement for confinement A_{sh} shall not be less than that given by the following two equations: $A_{sh} = 0.3(sh_c f'_c / f_{yh})[(A_g/A_{ch}) - 1]$ (23-3) $A_{sh} = 0.09sh_c f'_c / f_{yh}$ (23-4)	23.4.4.1(b)	8
Transverse reinforcement shall be provided by either single or overlapping hoops. Crossties of the same bar size and spacing as the hoops are permitted, with each end of the crosstie engaging a peripheral longitudinal reinforcing bar. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement.	23.4.4.1 (c)	
Eqs. (23-3) and (10-6) need not be satisfied if the design strength of the member core satisfies the requirement of the design loading combinations, including the earthquake effects.	23.4.4.1(d)	—
If the thickness of the concrete outside of the confining transverse reinforcement > 100 mm, additional transverse reinforcement shall be provided at a spacing ≤ 300 mm. Concrete cover on the additional reinforcement ≤ 100 mm.		

Table 10 - Transverse Reinforcement Requirements (2)

	Sect. No.	Fig. No.
Transverse reinforcement shall be spaced at distances not exceeding: • Minimum member dimension/4 • 6 × longitudinal bar diameter • s_x where $100\text{ mm} \le s_x = 100 + \frac{350 - h_x}{3} \le 150\text{ mm}$. (23-5)	23.4.4.2	8
Crossties or legs of overlapping hoops shall not be spaced more than 350 mm. on center in the direction perpendicular to the longitudinal axis of the structural member. Vertical bars shall not be farther than 150 mm. clear from a laterally supported bar.	23.4.4.3 7.10.5.3	
Where transverse reinforcement as required in 23.4.4.1-23.4.4.3 is no longer required, the remainder of the column shall contain spiral or hoop reinforcement spaced at distances not to exceed: • 6 × longitudinal bar diameter • 150 mm.	23.4.4.6	
Transverse reinforcement must also be proportioned to resist the design shear forces.	23.4.5	
Columns supporting reactions from discontinued stiff members, such as walls, shall have transverse reinforcement as specified in 23.4.4.1-23.4.4.3 over their full height, if the factored axial compressive force related to earthquake effects > $A_g f'_c/10$. This transverse reinforcement shall extend into the discontinued member for at least the development length of the largest longitudinal reinforcement in the column in accordance with 23.5.4. • If the lower end of the column terminates on a wall, transverse reinforcement per 23.4.4.1-23.4.4.3 shall extend into the wall for at least the development length of the largest longitudinal bar in the column at the point of termination. • If the column terminates on a footing or mat, transverse reinforcement per 23.4.4.1-23.4.4.3 shall extend at least 300 mm. into the footing or mat.	23.4.4.5	10

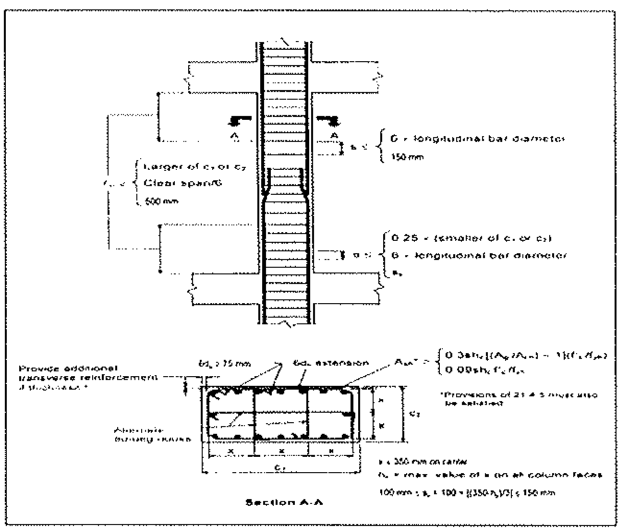


Figure 8 - Transverse Reinforcement Requirements - Rectangular Hoop Reinforcement

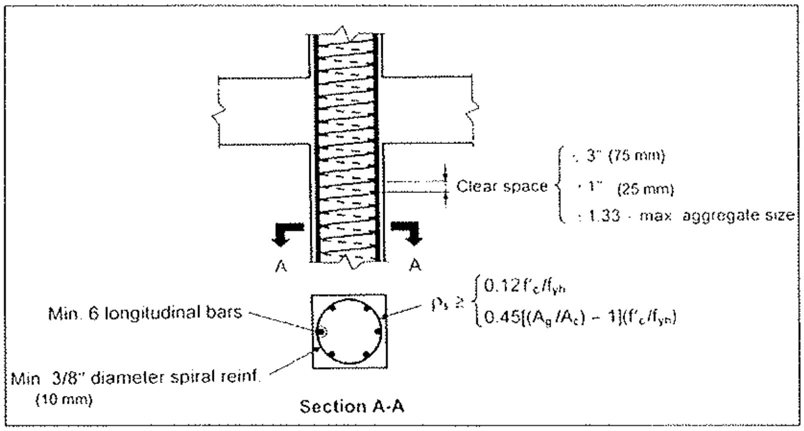


Figure 9 - Transverse Reinforcement Requirements - Spiral Hoop Reinforcement

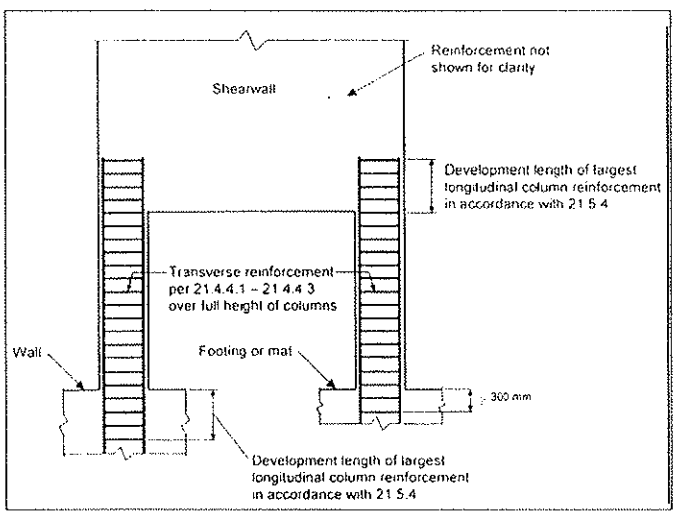


Figure 10 - Columns Supporting Discontinued Stiff Members

3. Joints of Special Moment Frames

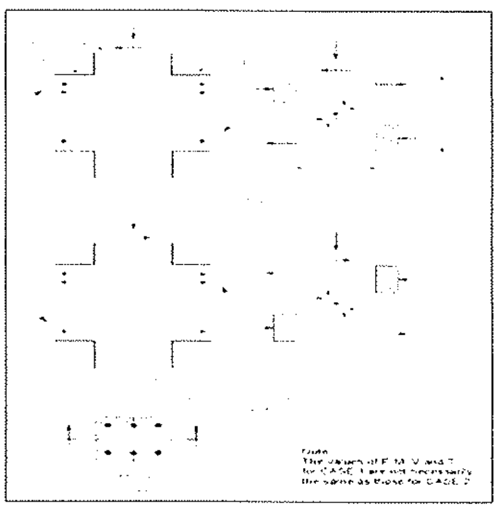
General Requirements

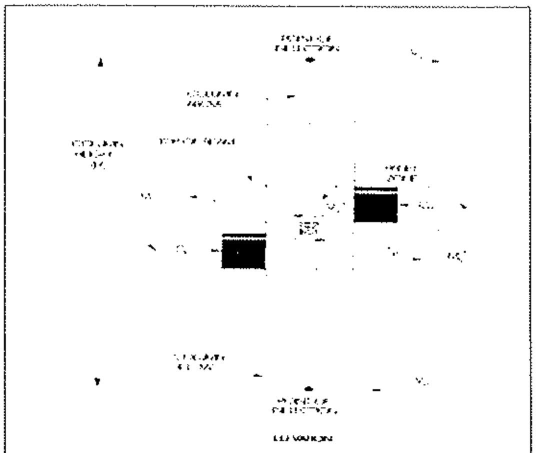
- The overall integrity of special moment frames, which are required in regions of high seismic risk or for structures assigned to high seismic performance or design categories, is dependent on the behavior of beam-column joints. Degradation of joints can result in large lateral deformations, which can cause excessive damage or even failure. The general requirements of 23.5.1 are summarized in Table 11 and Fig. 11.

- Since the development of inelastic rotations at the faces of the joints is associated with strains in the flexural reinforcement significantly greater than the yield strain, joint shear forces generated by the flexural reinforcement are calculated based on a stress in the reinforcement equal to 1.25*fy* (23.5.1.1).

- Slippage of longitudinal reinforcement in a beam column joint can lead to an increase in the joint rotation. Longitudinal bars must be continued through the joint or must be properly developed for tension and compression in the confined column core (23.5.1.3). The minimum column size requirements of 23.5.1.4 reduce the possibility of failure from loss of bond, considering load reversals beyond the yield point of the steel that are anticipated during a major earthquake.

Beam-Column Joint Analysis





Column Shear Force V_u

Transverse Reinforcement

- Transverse reinforcement in a beam-column joint is required to adequately confine the concrete to ensure its ductile behaviour and to allow it to maintain its vertical loadcarrying capacity even after possible spalling of the outer shell. The transverse reinforcement requirements of 23.5.2 for joints of special moment frames are summarized in Table 12. Minimum confinement reinforcement equal to the amount specified in 23.4.4 for potential hinging regions in columns must be provided within a joint, unless the joint is confined by structural members per 23.5.2.2. Figure 11 illustrates the requirements when less than four members frame into a beam-column joint.

- Fifty percent of the confining reinforcement required by 23.4.4 may be used when members frame into all four sides of a joint, provided the width of the member is at least three-fourths the corresponding column width. This reduction in the amount of transverse reinforcement recognizes the beneficial effect provided by these members in resisting bursting pressures that can be generated within a joint. The requirements of 23.5.2,2 are shown in Fig. 12.

- Section 23.5.2.3 contains provisions for joints where the beam width is greater than the corresponding column width. Beam reinforcement that is not confined by column reinforcement shall be confined by transverse reinforcement per 23.4.4, unless a beam framing into the joint provides confinement (see Fig. 13).

- The minimum amount of transverse reinforcement for all of the cases noted above must be provided through the joint regardless of the magnitude of the calculated shear force in the joint.

Table 11 - General Requirements

	Sect. No.	Fig. No.
Forces in longitudinal beam reinforcement at the face of the joint shall be determined assuming that the stress in the flexural tensile reinforcement is equal to 1.25*fy*.	23.5.1.1	-
Strength of joint shall be governed by the appropriate strength reduction factors in 9.3.	23.5.1.2	-
Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored • In tension according to 23.5.4 • In compression according to Chapter 12	23.5.1.3	11
Where longitudinal beam reinforcement extends through a beam-column joint, column dimension parallel to the beam reinforcement shall not be less than • 20 × diameter of the largest longitudinal bar for normal weight concrete • 26 × diameter of the largest longitudinal bar for lightweight concrete	23.5.1.4	

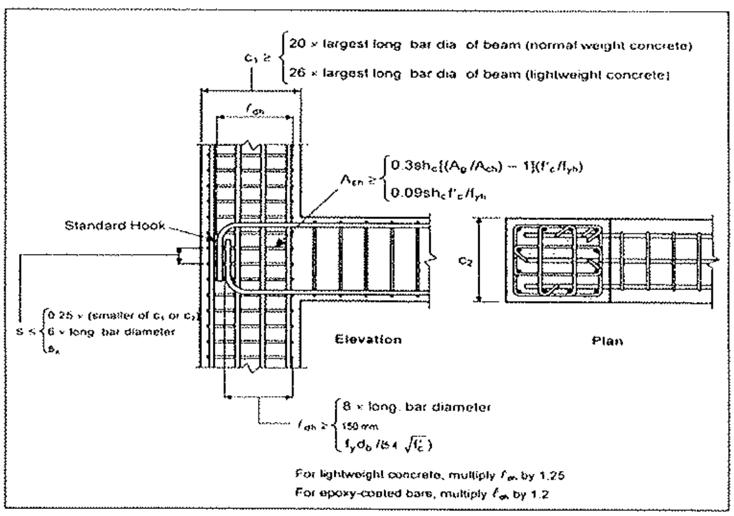


Figure 11 -General Requirements and Transverse Reinforcement Requirements for Joints not Confined by Structural Members

Table 12 - Transverse Reinforcement Requirements

	Sect. No.	Fig. No.
Transverse hoop reinforcement required for column ends per 23.4.4 shall be provided within a joint, unless structural members confine the joint as specified in 23.5.2.2.	23.5.2.1	11
Where members frame into all four sides of a joint and each member width is at least 3/4 the column width, the transverse reinforcement with in the depth of the shallowest member may be reduced to 1/2 of the amount required by 23.4.4.1. The spacing of the transverse reinforcement required in 23.4.4.2(b) shall not exceed 150 mm at these locations.	23.5.2.2	12
Transverse reinforcement per 23.4.4 shall be provided through the joint to confine longitudinal beam reinforcement outside the column core if a beam framing into the joint does not provide such confinement.	23.5.2.3	13

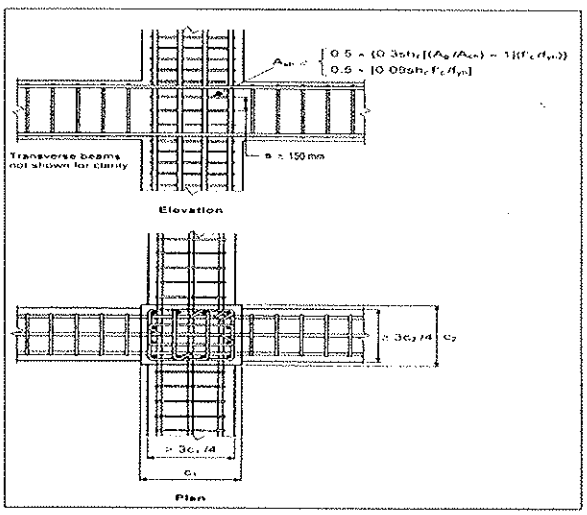


Figure 12 -Transverse Reinforcement Requirements for Joints Confined by Structural Members

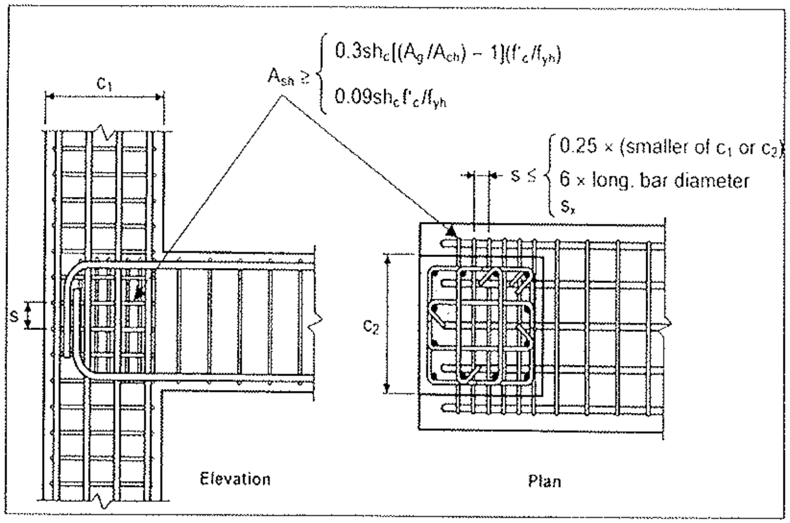


Figure 13 -Transverse Reinforcement Requirements for Longitudinal Beam Reinforcement Outside a Confined Column Core

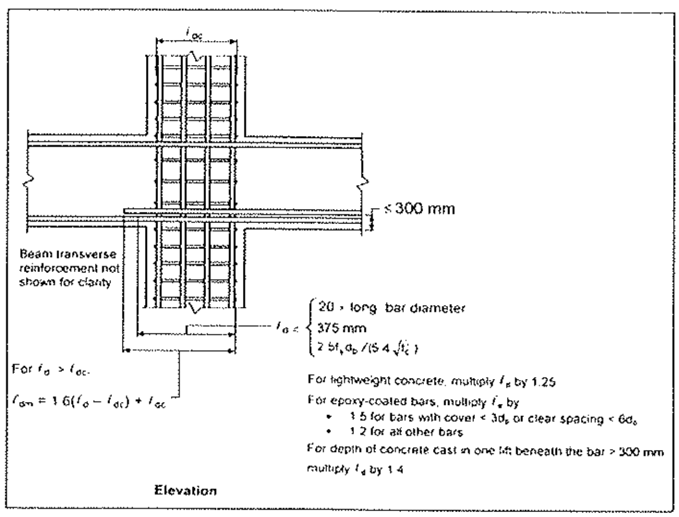


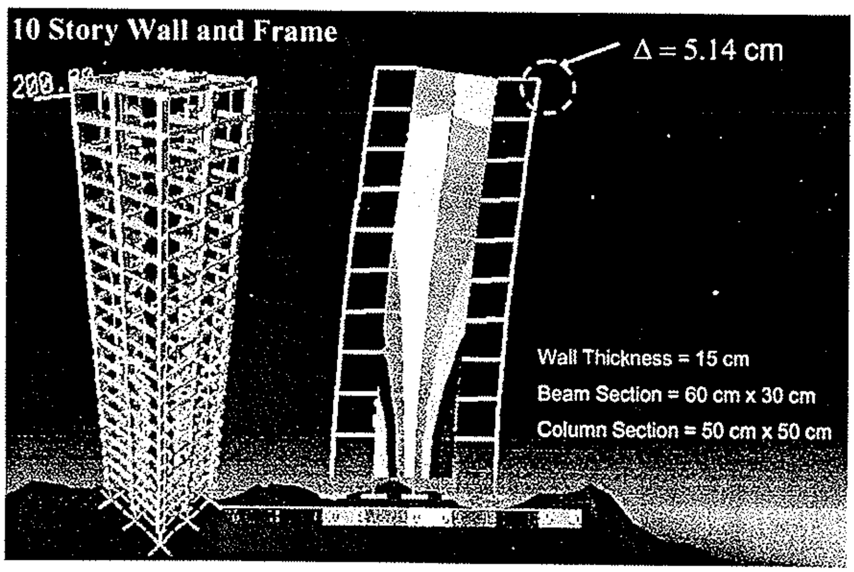
Figure 14 - Development Length of Straight Bars in Tension

Special Reinforced Concrete Structural Walls and Coupling Beams

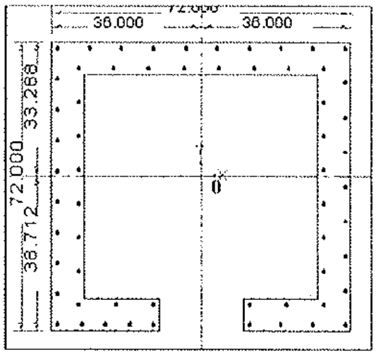
Scope

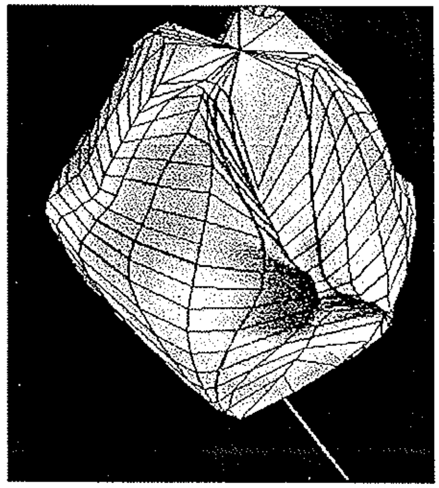
The provisions of 23.6 apply to special reinforced concrete structural walls and coupling beams that are part of the earthquake force-resisting system. These provisions are required in regions of high seismic risk or for structures assigned to high seismic performance and design categories.

Shear Wall–Frame Interaction

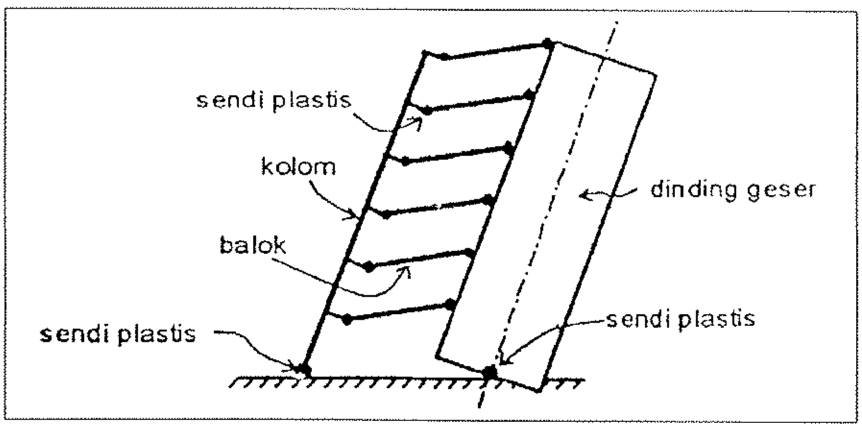


Interaction Surface for Shear Walls





Special Reinforced Concrete Structural Walls Behaviour



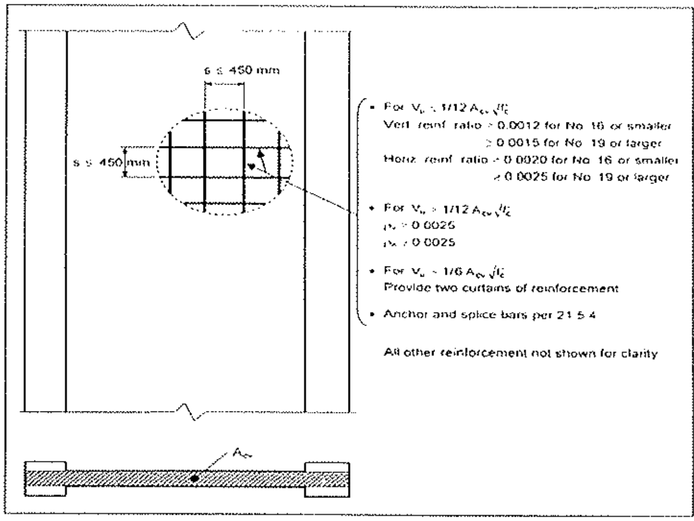


Figure 15 - Web Reinforcement Requirements

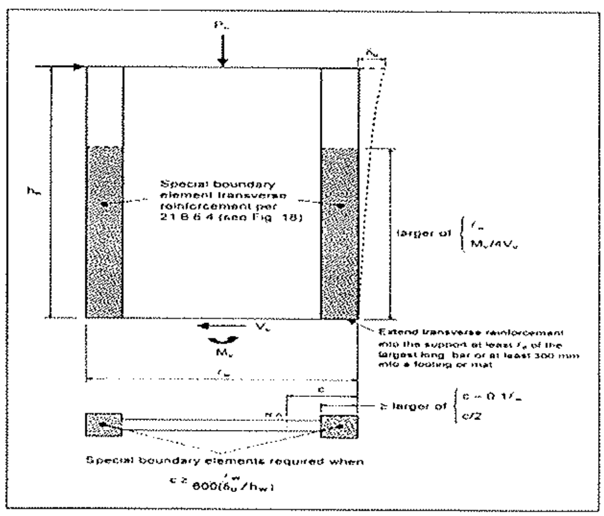


Figure 16 - Boundary Element Requirements per 23.6.6.2

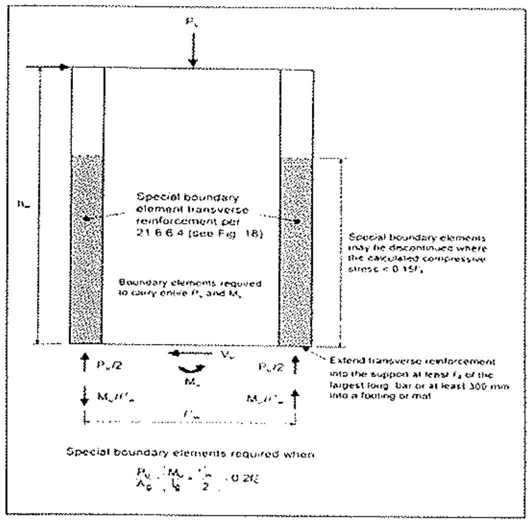


Figure 17 - Boundary Element Requirements per 23.6.6.3

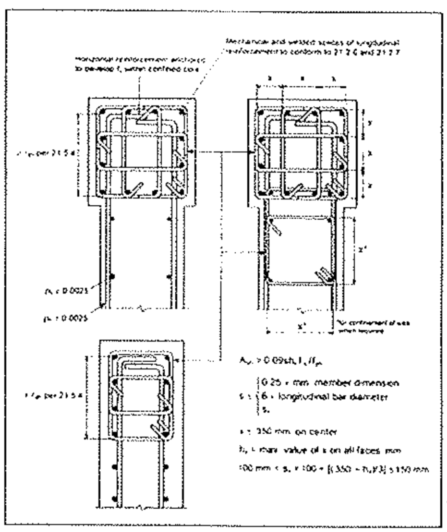


Figure 18 - Reinforcement Details for Boundary Element

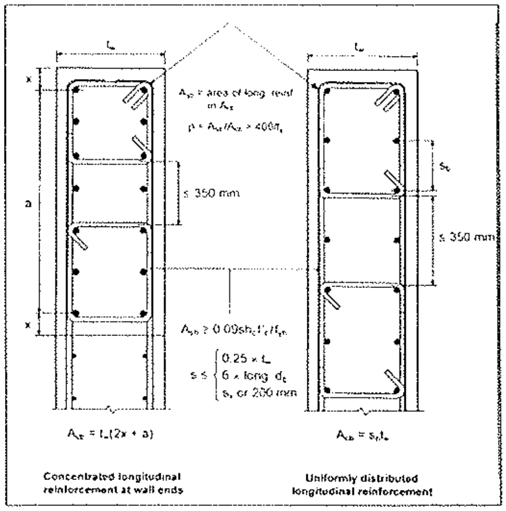


Figure 19 - Reinforcement Details where Boundary Elements are Not Required

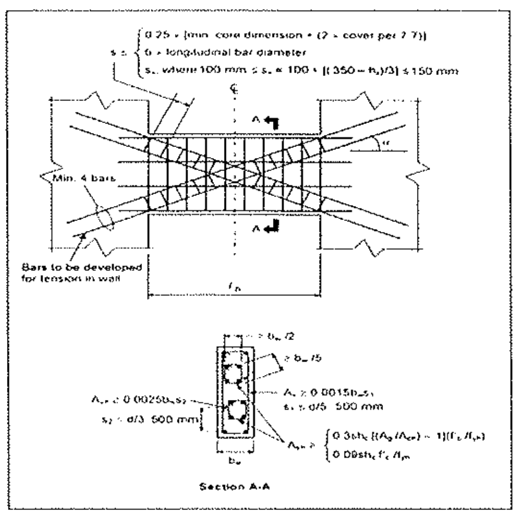


Figure 20 - Coupling Beam Requirements per 23.6.7.4

Foundations

Scope

Requirements for foundations supporting structures in regions of high seismic risk or for structures assigned to high seismic performance or design categories are contained in 23.8. Provisions for piles, caissons, and slabs on grade supplement other design and construction criteria in SNI 03-2847-2002.

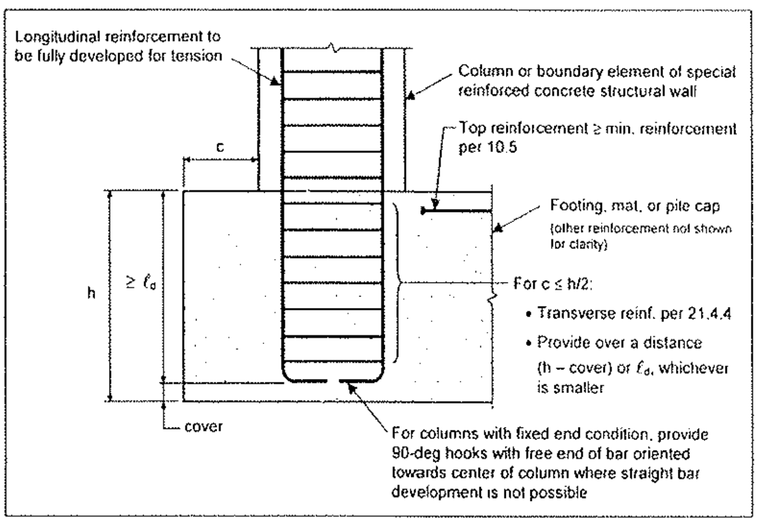


Figure 21 - Requirements for Footings, Foundation Mats, and Pile Caps

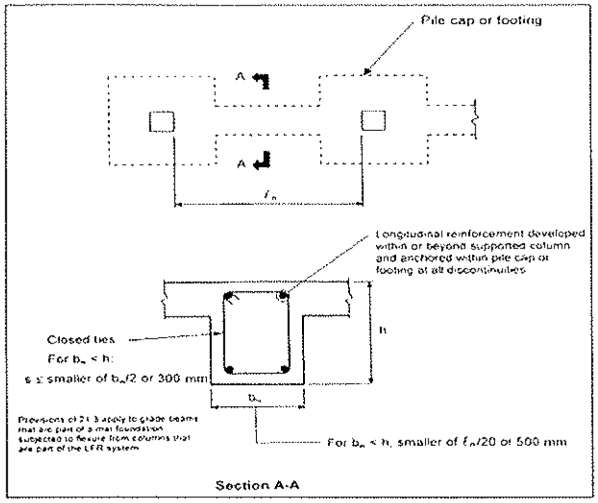


Figure 22 - Requirements for Grade Beams

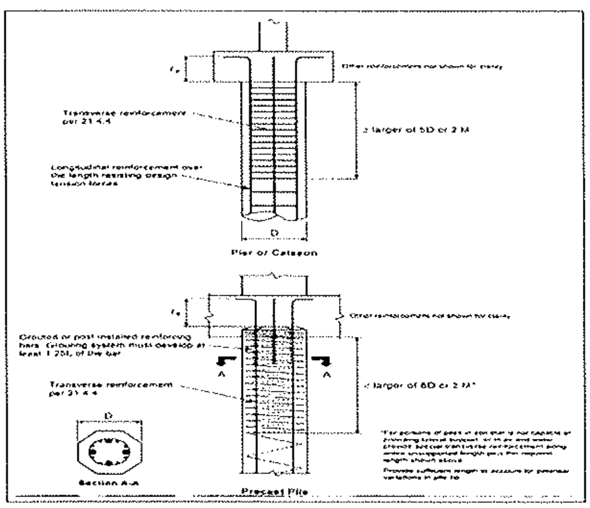


Figure 23 - Requirements for Piles, Piers, and Caissons

Intermediate Moment Frames

General Requirements

- Provisions for intermediate moment frames, which are required in regions of moderate seismic risk or for structures assigned to intermediate (moderate) seismic performance or design categories, are given in 23.10.
- Two options are provided in 23.10.3 to determine the design shear strength of beams, columns, and two-way slabs, both of which reduce the likelihood of shear failure during an earthquake.

Beams

The requirements of 23.10.4 for beams of intermediate moment frames are summarised in, Fig. 24, and Fig. 25.
The main purpose of these requirements is to provide beams with a threshold level of toughness

Columns

Fig. 26 contain the detailing requirements of 23.10.5 for columns of intermediate moment frames. Similar to the requirements for beams, these provisions provide columns with a threshold level of toughness.

Two-way Slabs without Beams

Two-way slabs without beams are acceptable lateral-force resisting systems in regions of low or moderate seismic risk, or for structures assigned to low or intermediate seismic performance or design categories. They are not permitted to be part of the lateral-force-resisting system in regions of high seismic risk, or for structures assigned to high seismic performance or design categories. Figure27 illustrates the detailing requirements of 23.10.6.1 through 23.10.6.3 at support locations. The moment M_s is that portion of the factored slab moment that is balanced by the supporting members at a joint for a given load combination that includes earthquake effects. It is not necessarily equal to the total design moment at a support.
Detailing requirements for column strips and middle strips are contained in Figs. 28 and 29, respectively.

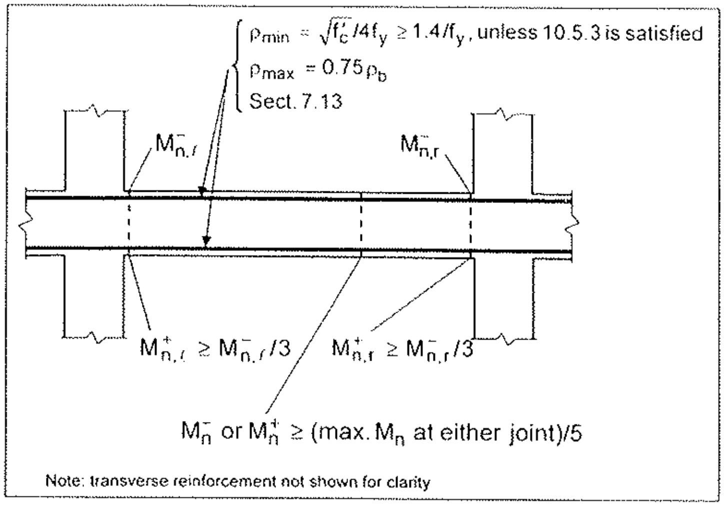


Figure 24 - Flexural Requirements for Beams

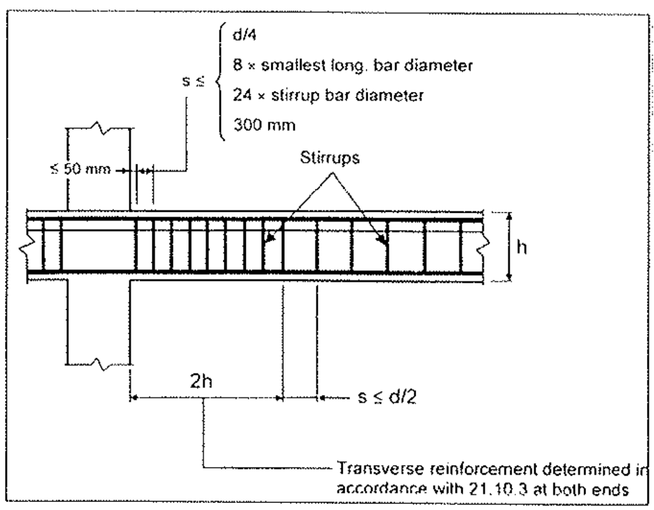


Figure 25 - Transverse Reinforcement Requirements for Beams

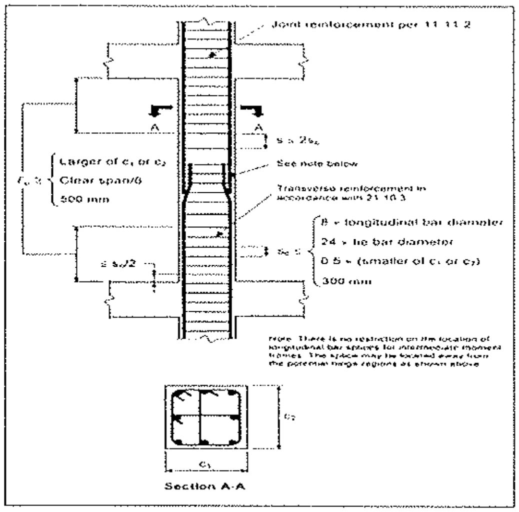


Figure 26 - Transverse Reinforcement Requirements for Columns

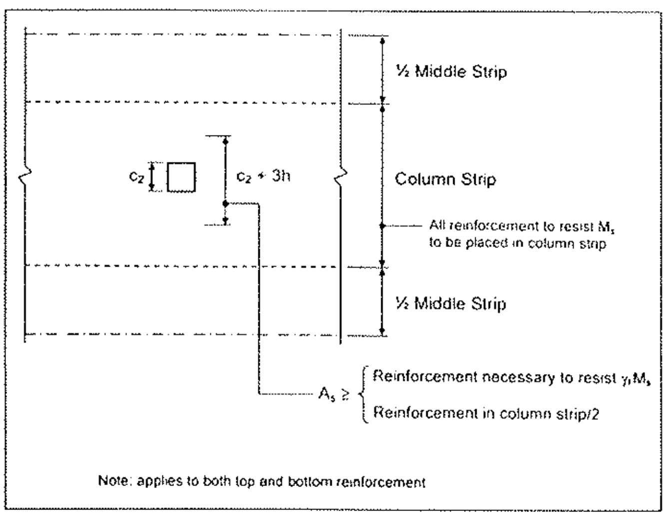


Figure 27 - Reinforcement Details at Supports of Two-way Slabs without Beams

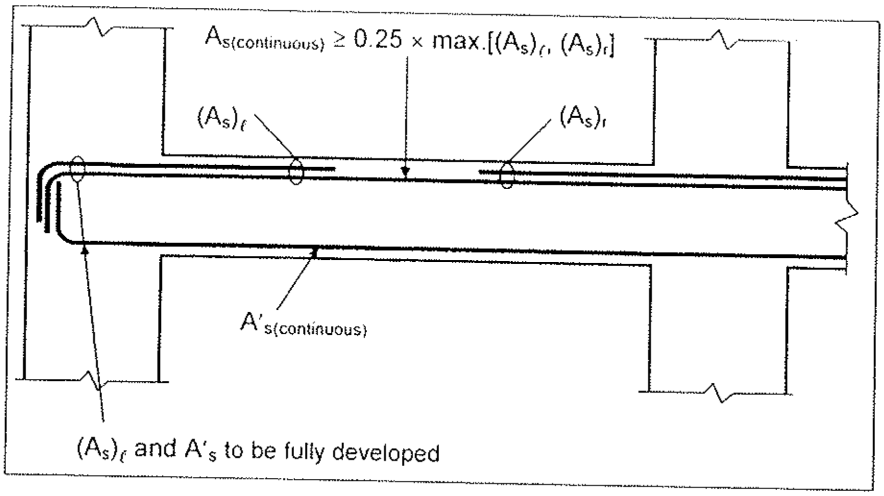


Figure 28 -Reinforcement Details in Two-way Slabs without Beams: Column Strip

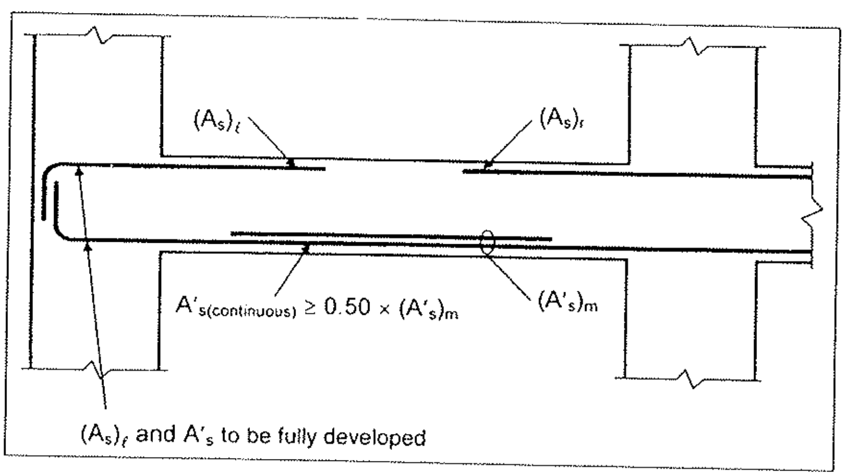


Figure 29 -Reinforcement Details in Two-way Slabs without Beams: Middle Strip

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Thank you