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Casa abierta al tiempo
Azcapotzalco

- **EDUCACION**
  - Licenciatura en Ingeniería Civil, Universidad Autónoma Metropolitana (UAM)
  - Maestría en Estructuras, Universidad de Texas en Austin
  - Doctorado en Estructuras, Universidad de California en Berkeley

- **VARIOS**
  - Profesor-investigador, UAM
  - Mesa Directiva SMIS, 1999-2001, 2004-2005
  - Mesa Directiva SMIE, 2003-2004
  - Miembro fundador del Consejo Consultivo sobre Sismos
  - Mas de 100 publicaciones, la mayoría relacionada con Ingeniería Sísmica.
  - Actividades desempeñadas en el pasado
    - Jefe del Área de Estructuras UAM
    - Asesoría a despachos de cálculo estructural (Colinas de Buen)

- **DISTINCIONES**
  - Premio Jose A. Cuevas, otorgado por el Colegio de Ingenieros Civiles de México.
  - Medalla al Mérito Universitario, otorgada por la Universidad Autónoma Metropolitana
  - Investigador Nacional, otorgado por el Sistema Nacional de Investigadores
  - Miembro Titular de la Academia Nacional de Ingeniería
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Evaluación Estructural Basada en Desplazamientos

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- **Objetivo: Evaluar si las propiedades estructurales de una edificación son capaces de controlar adecuadamente su nivel de daño estructural de acuerdo a sus objetivos de diseño.**

- **Alcance: Se han llevado una serie de análisis preliminares que apuntan hacia la necesidad de hacer una evaluación mas refinada que permita establecer la necesidad de rehabilitar (reparar o reforzar) la edificación.**

Tendencias actuales del diseño sísmico

Paradigma (Kuhn/Capra):

Constelación de logros-conceptos, valores, percepciones, técnicas y prácticas-compartidos por una comunidad ingenieril, que conforman una particular visión de la realidad que a su vez, da lugar a la base que le permite plantear y definir proyectos y sus soluciones legítimas.

Existe un cambio de paradigma. El nuevo paradigma debe admitir que todos los conceptos y teorías son limitados y aproximados. Los ingenieros nunca tratan con la verdad, sino con descripciones aproximadas de la realidad. Bajo este contexto, es necesario atender mas al aspecto conceptual del problema, y utilizar un enfoque sistémico.

Con lo anterior en mente, el diseño sísmico puede plantearse como un problema de demanda-capacidad:

DEMANDA SÍSMICA ≤ CAPACIDAD SÍSMICA

Objetivos de diseño de una estructura de ocupación estándar:

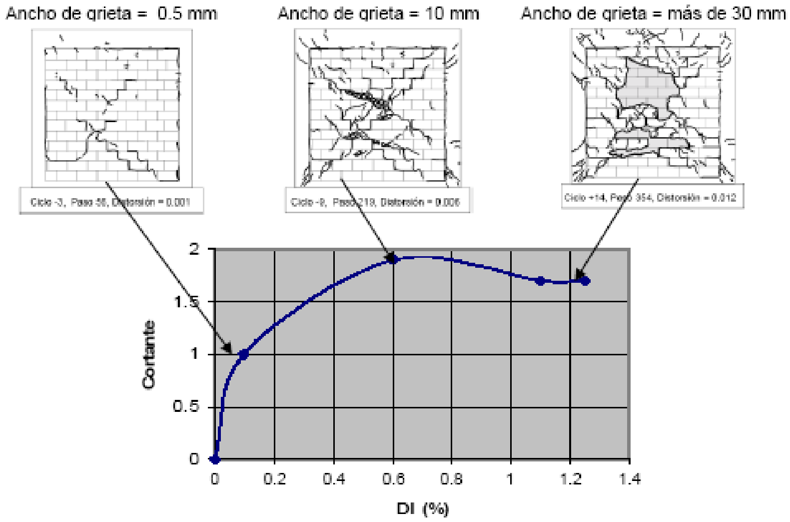
- **Resistir sin daño niveles menores de movimiento sísmico**
- **Resistir sin daño estructural, aunque posiblemente con algún tipo de daño no estructural, niveles moderados de movimiento sísmico**
- **Resistir sin colapso, aunque con algún tipo de daño estructural y no estructural, niveles mayores de movimiento sísmico**

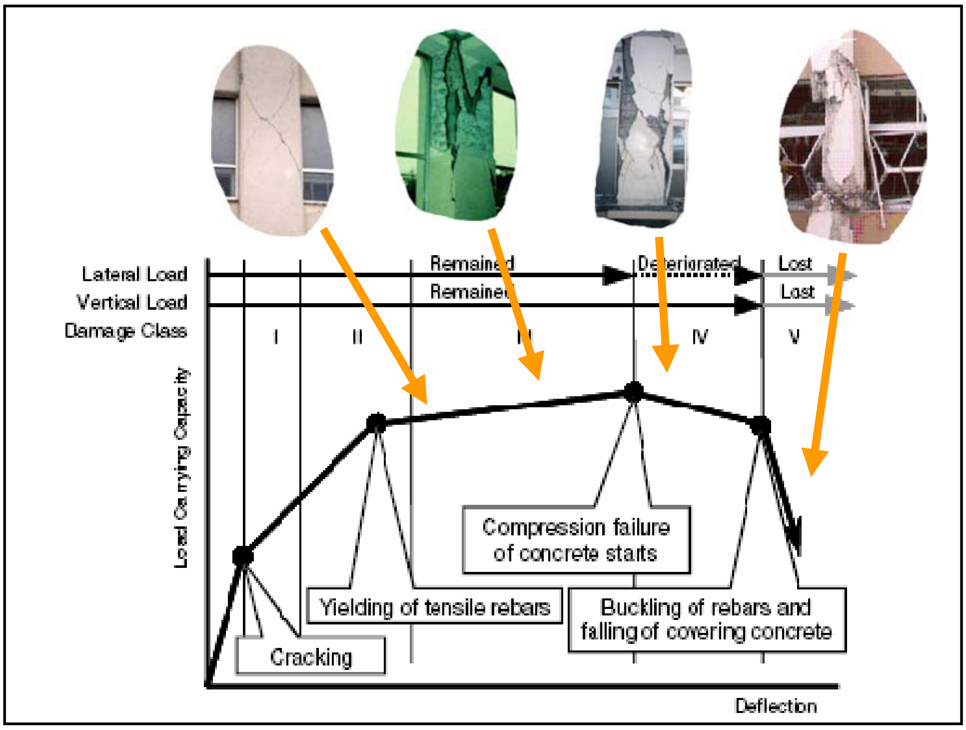
DEMANDA SÍSMICA ≤ CAPACIDAD SÍSMICA

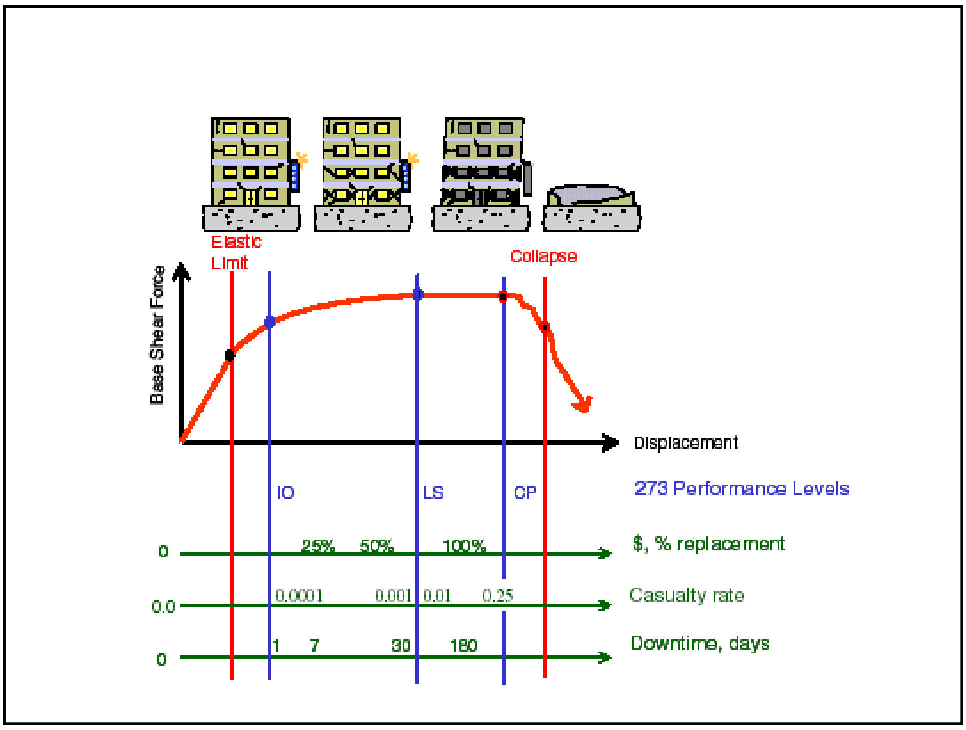
de	***de***
Resistencia	***Resistencia***
Rigidez	***Rigidez***
Capacidad de Deformación	***Capacidad de Deformación***

El nivel de daño o de degradación que sufren los elementos estructurales, no estructurales y el contenido dependen de los valores del desplazamiento lateral (deformación plástica), velocidad, aceleración.

Un menor nivel de respuesta implica menor nivel de daño


Ancho de grieta = 0.5 mm
Ancho de grieta = 10 mm
Ancho de grieta = más de 30 mm
Cortante
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
1.2
1.4
DI (%)


Lateral Load
Vertical Load
Damage Class
Remained
Remained
Deteriorated
Lost
Lost
I
II
III
IV
V
Load Carrying Capacity
Compression failure of concrete starts
Yielding of tensile rebars
Buckling of rebars and falling of covering concrete
Cracking
Deflection


Elastic Limit
Collapse
Base Shear Force
Displacement
IO
LS
CP
273 Performance Levels
0
25%
50%
100%
$, % replacement
0.0
0.0001
0.001
0.01
0.25
Casualty rate
0
1
7
30
180
Downtime, days

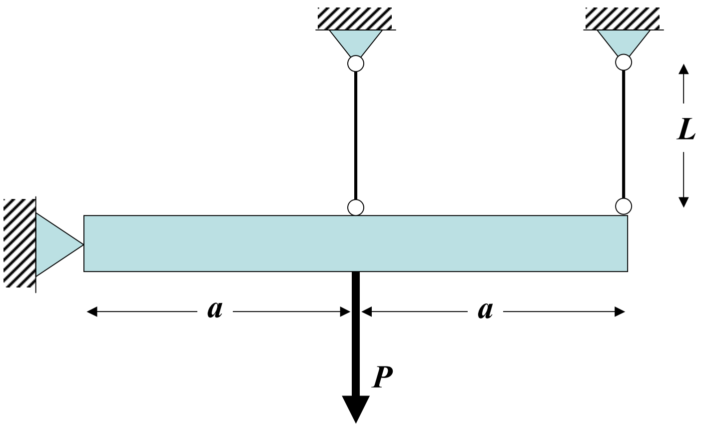
Las características mecánicas de la estructura deben proporcionarse para controlar (rigidez, resistencia, disipación de energía) y acomodar (capacidad de deformación), dentro de límites técnicos y económicos aceptables, su respuesta dinámica durante las excitaciones sísmicas de diseño

Con sistemas estructurales tradicionales es posible controlar la demanda de desplazamiento lateral (rigidez y resistencia), mientras que el control razonable de la velocidad y la aceleración solo es posible por medio de sistemas innovadores (disipación extra de energía, aislamiento).

Las nuevas tendencias de diseño sísmico demandan del ingeniero estructural el manejo explícito de las características mecánicas de diferente tipo de sistemas estructurales con el fin de controlar adecuadamente la respuesta dinámica de la estructura.

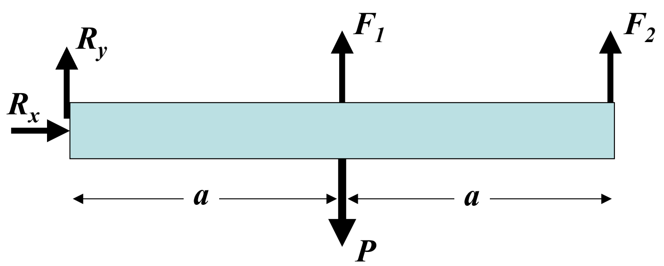
Conceptos

Considere el análisis de la siguiente estructura:



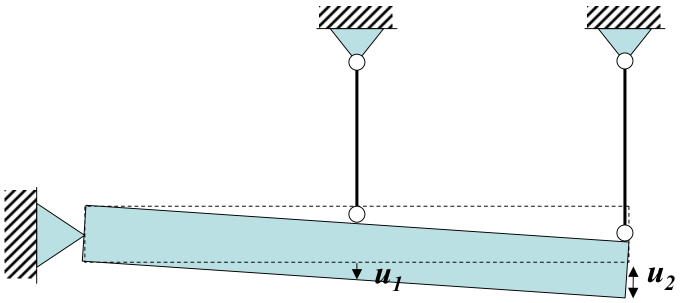
- ***AE* para ambas barras**
- **Viga rígida**

Equilibrio:



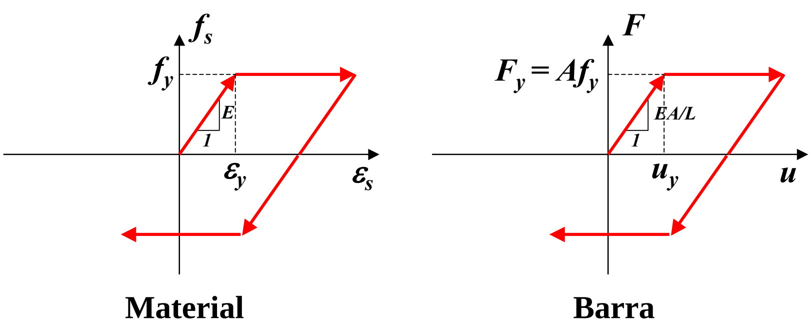
$$F_1 + 2F_2 = P$$

Compatibilidad:

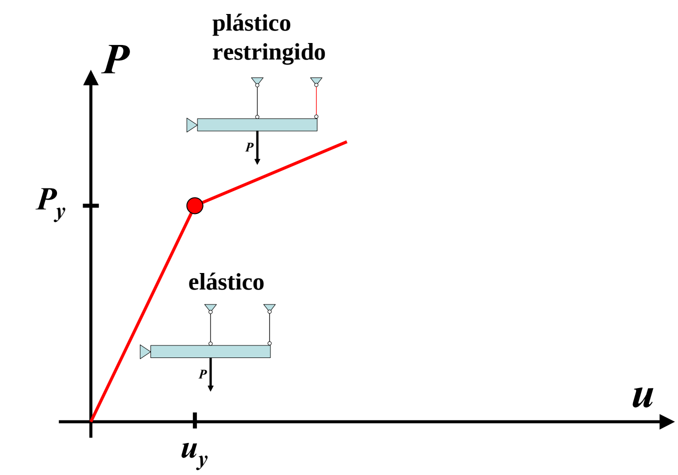


$$u_2 = 2u_1$$

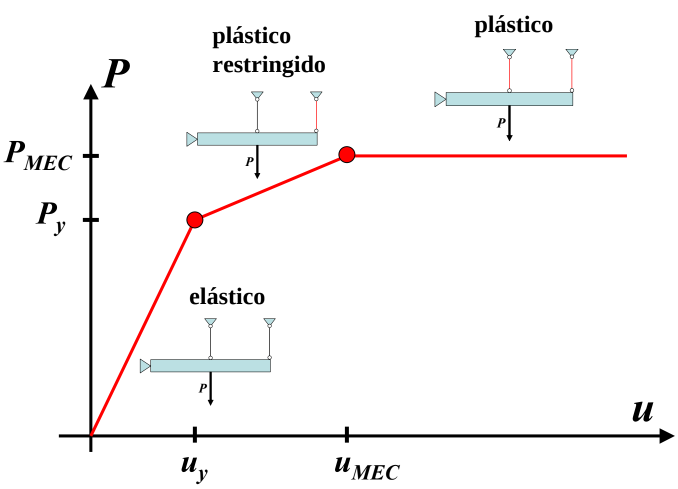
Constitutivas:



Primera Fluencia: $F_2 = Af_y$



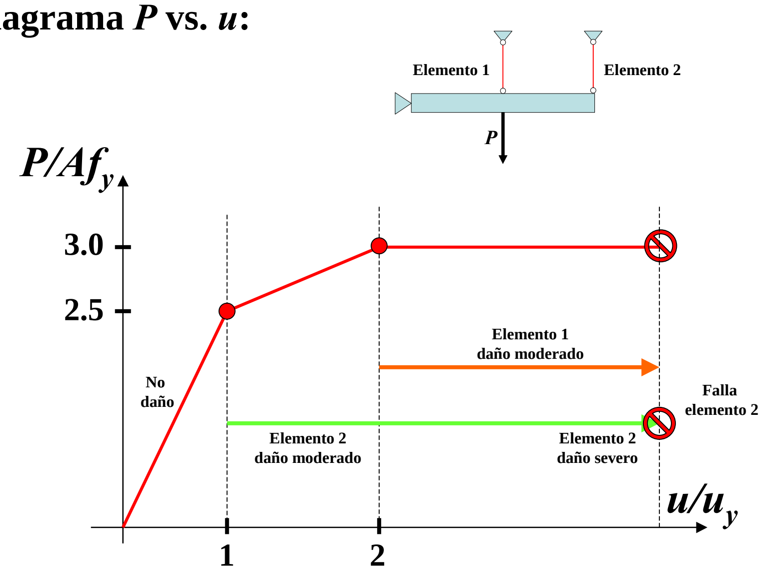
Formación de mecanismo: $F_1 = F_2 = Af_y$



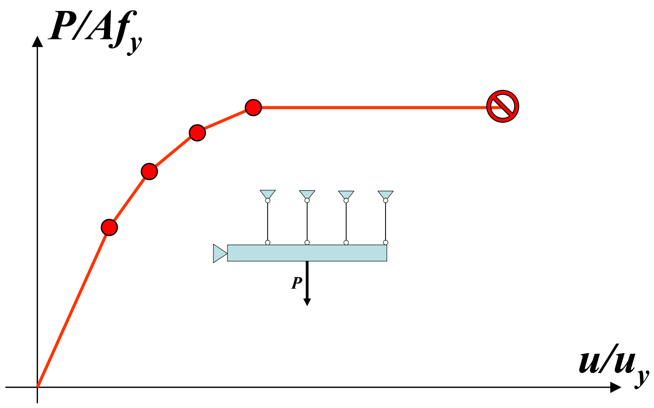
Daño estructural:

- **El nivel de daño estructural que sufre un elemento depende de su nivel de deformación plástica.**
- **Conforme mayor sea la demanda de deformación plástica, mayor será el nivel de daño.**

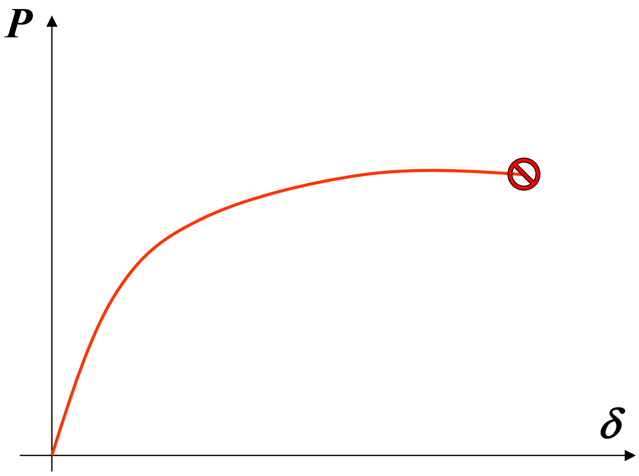
Diagrama *P* vs. *u*:



Conforme crece el número de elementos:



En el límite:



Observaciones:

- **La resistencia última de una estructura puede ser significativamente mayor que aquella asociada a la primera fluencia ($P_{MEC} > P_y$).**
- **Es la fluencia gradual de la estructura, que depende del grado de indeterminación estática, la que permite el incremento paulatino desde P_y hasta P_{MEC}.**

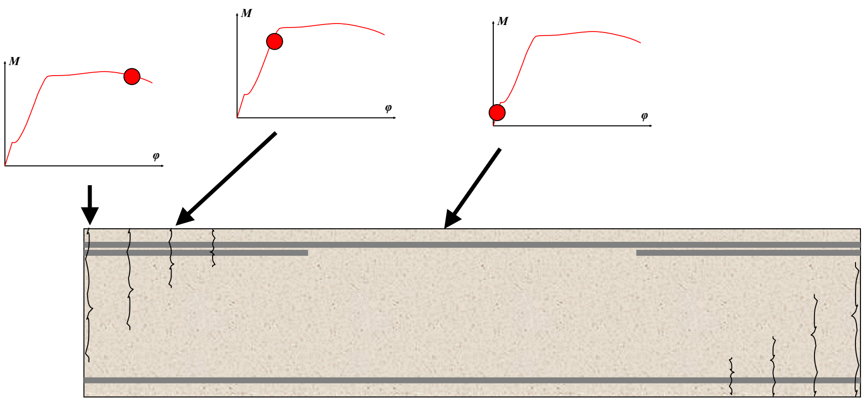
Observaciones:

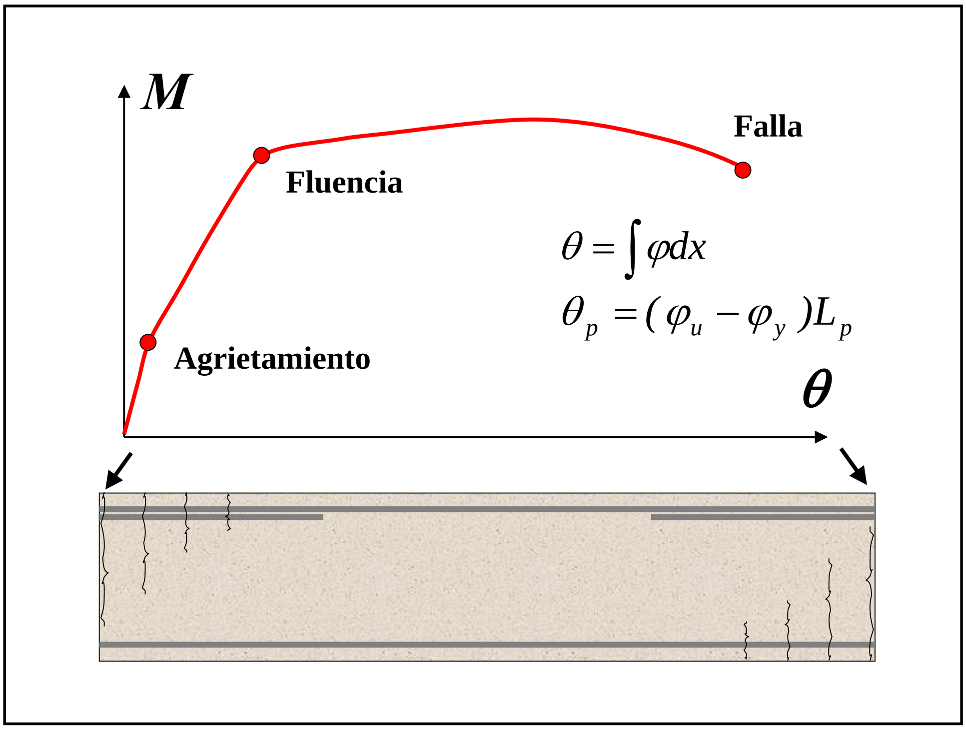
- **Un elemento estructural que fluye no es capaz de acomodar un mayor nivel de carga.**
- **Si los elementos que fluyen son capaces de deformarse en el rango plástico de comportamiento, los elementos que permanecen elásticos contribuyen al incremento de la capacidad resistente de la estructura.**

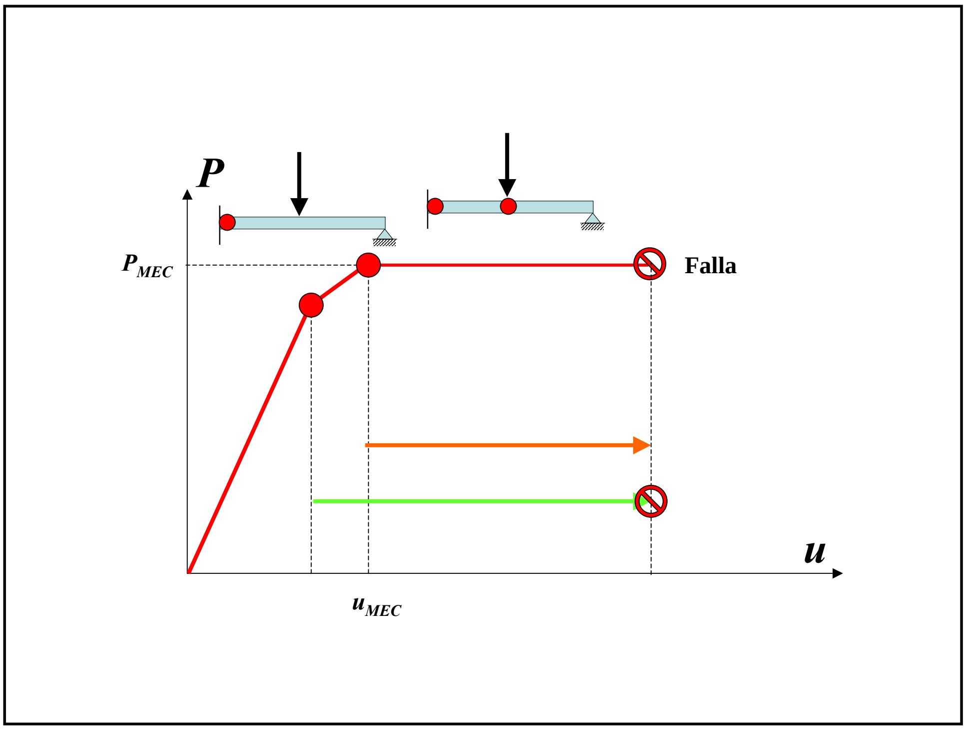
Observaciones:

- **El comportamiento plástico en nuestras estructuras estructuradas con base en marcos momento-resistentes suele concentrarse como rotaciones plásticas que tienden a concentrarse en sus extremos. Para evaluar el daño bajo estas circunstancias, suele emplearse el concepto de articulación plástica y de capacidad rotacional.**

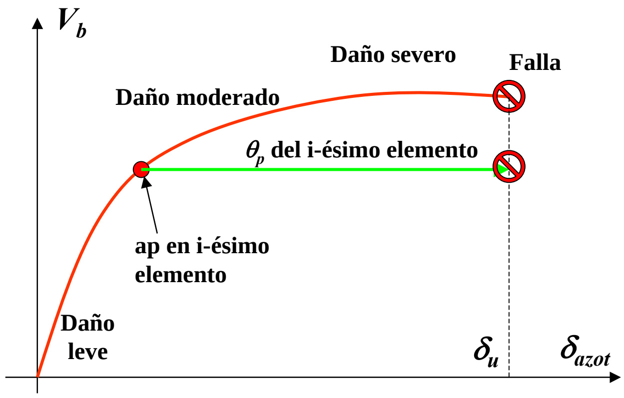
Sección {
No agrietada
Agrietada
Fluencia moderada
Fluencia significativa
Geometría/Acero




M
Falla
Fluencia
$\theta = \int \varphi dx$
$\theta_p = (\varphi_u - \varphi_y) L_p$
Agrietamiento
θ


P
P_{MEC}
Falla
u
u_{MEC}

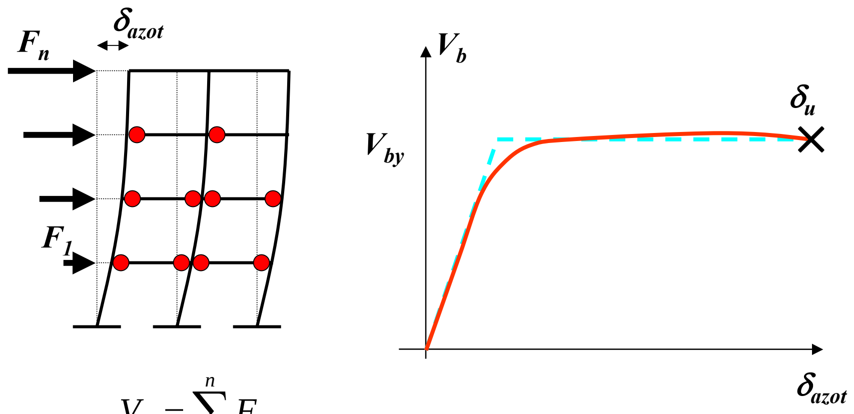
Nivel Estructura:



Evaluación estructural basada en desplazamientos para edificaciones sismorresistentes:

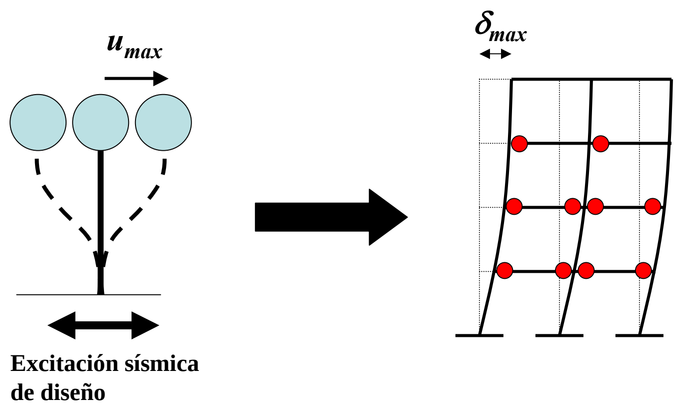
- **Realizar un análisis estático no lineal de la estructura para definir su curva *P* vs. δ.**
- **Estimar la máxima demanda de desplazamiento (δ_{max}) que en la estructura induce la excitación sísmica de interés.**
- **En función de valor de δ_{max}, revisar si el estado de daño en los elementos estructurales es consistente con sus objetivos de diseño.**

1) Análisis estático no lineal:

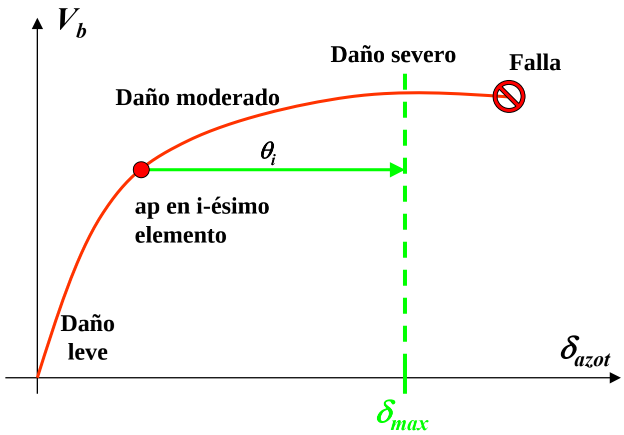


$$V_b = \sum_{i=1}^{n} F_i$$

2) Estimación de u_{max} (análisis dinámico):



3) Evaluación de nivel de daño:



FEMA 306. Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings

Definición de excitaciones sísmicas de interés

	Earthquake Hazard Level	Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
		Building Performance Levels			
Leve	50%/50 year	a	b	c	d
Moderado	20%/50 year	e	f	g	h
Severo	BSE-1 (~10%/50 year)	i	j	k	l
Extremo	BSE-2 (~2%/50 year)	m	n	o	p

k + p = BSO
k + p + any of a, e, i, m; or b, f, j, or n = Enhanced Objectives
o = Enhanced Objective
k alone or p alone = Limited Objectives
c, g, d, h = Limited Objectives

Definición de objetivos de diseño :

Earthquake Hazard Level	Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
	Building Performance Levels			
50%/50 year	a	b	c	d
20%/50 year	e	f	g	h
BSE-1 (~10%/50 year)	i	j	k	l
BSE-2 (~2%/50 year)	m	n	o	p

k + p = BSO
k + p + any of a, e, i, m; or b, f, j, or n = Enhanced Objectives
o = Enhanced Objective
k alone or p alone = Limited Objectives
c, g, d, h = Limited Objectives

Definición de objetivos de diseño (general):

	Building Performance Levels			
	Collapse Prevention Level	**Life Safety Level**	**Immediate Occupancy Level**	**Operational Level**
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Building is near collapse.	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	No permanent drift; structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional.
Nonstructural components	Extensive damage.	Falling hazards mitigated but many architectural, mechanical, and electrical systems are damaged.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.	Negligible damage occurs. Power and other utilities are available, possibly from standby sources.
Comparison with performance intended for buildings designed, under the *NEHRP Provisions,* for the Design Earthquake	Significantly more damage and greater risk.	Somewhat more damage and slightly higher risk.	Much less damage and lower risk.	Much less damage and lower risk.

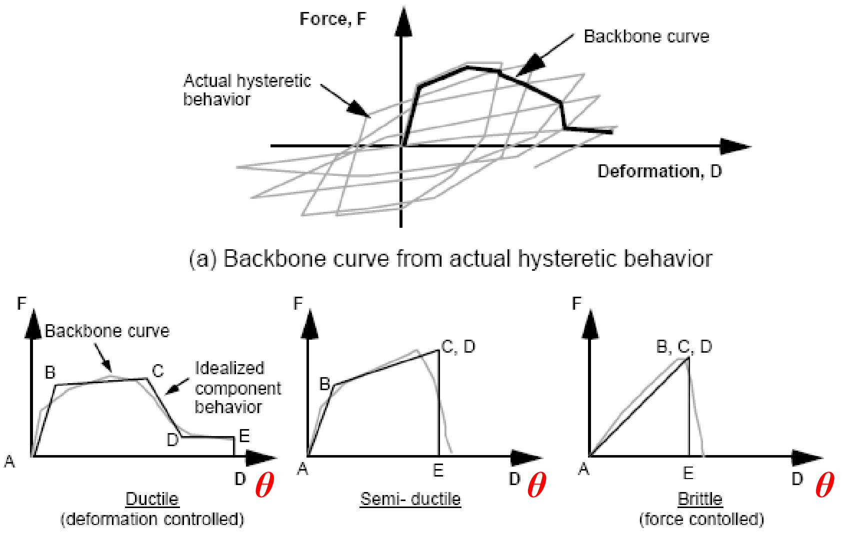
Definición de objetivos de diseño (estructural):

		Structural Performance Levels		
Elements	**Type**	**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" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks < 1/8" 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" width.
	Drift[2]	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent

Definición de objetivos de diseño (no estructural):

Component	Nonstructural Performance Levels: Hazards Reduced Level N-D	Life Safety N-C	Immediate Occupancy N-B	Operational N-A
Cladding	Severe damage to connections and cladding. Many panels loosened.	Severe distortion in connections. Distributed cracking, bending, crushing, and spalling of cladding elements. Some fracturing of cladding, but panels do not fall.	Connections yield; minor cracks (< 1/16" width) or bending in cladding.	Connections yield; minor cracks (< 1/16" width) or bending in cladding.
Glazing	General shattered glass and distorted frames. Widespread falling hazards.	Extensive cracked glass; little broken glass.	Some cracked panes; none broken.	Some cracked panes; none broken
Partitions	Severe racking and damage in many cases.	Distributed damage; some severe cracking, crushing, and racking in some areas.	Cracking to about 1/16" width at openings. Minor crushing and cracking at corners.	Cracking to about 1/16" width at openings. Minor crushing and cracking at corners.
Ceilings	Most ceilings damaged. Light suspended ceilings dropped. Severe cracking in hard ceilings.	Extensive damage. Dropped suspended ceiling tiles. Moderate cracking in hard ceilings.	Minor damage. Some suspended ceiling tiles disrupted. A few panels dropped. Minor cracking in hard ceilings.	Generally negligible damage. Isolated suspended panel dislocations, or cracks in hard ceilings.
Parapets and Ornamentation	Extensive damage; some fall in nonoccupied areas.	Extensive damage; some falling in nonoccupied areas.	Minor damage.	Minor damage.
Canopies & Marquees	Extensive distortion.	Moderate distortion.	Minor damage.	Minor damage.
Chimneys & Stacks	Extensive damage. No collapse.	Extensive damage. No collapse.	Minor cracking.	Negligible damage.
Stairs & Fire Escapes	Extensive racking. Loss of use.	Some racking and cracking of slabs, usable.	Minor damage.	Negligible damage.
Light Fixtures	Extensive damage. Falling hazards occur.	Many broken light fixtures. Falling hazards generally avoided in heavier fixtures (> 20 pounds).	Minor damage. Some pendant lights broken.	Negligible damage.
Doors	Distributed damage. Many racked and jammed doors.	Distributed damage. Some racked and jammed doors.	Minor damage. Doors operable.	Minor damage. Doors operable.

Definición de objetivos de diseño (estructural):



(b) Idealized component behavior from backbone curves

Definición de objetivos de diseño:

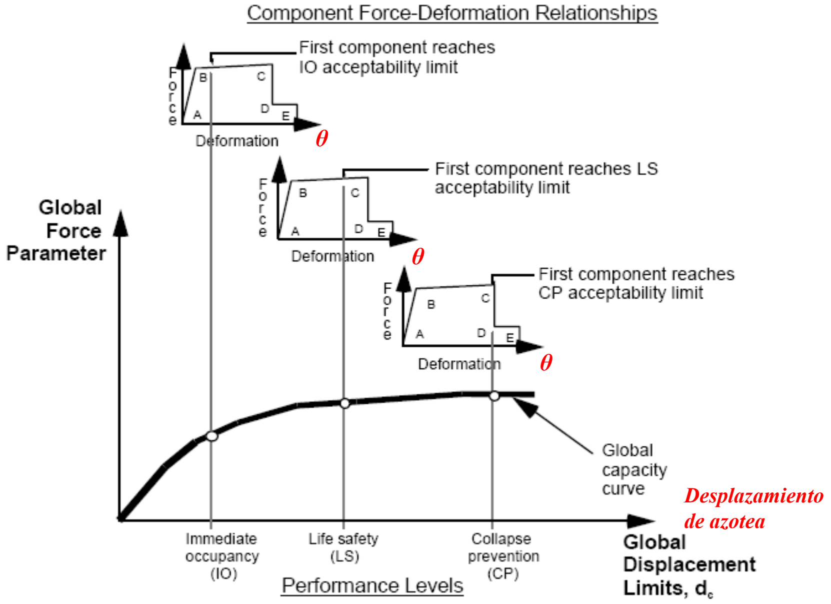


Table 6-6 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— Reinforced Concrete Beams

Conditions			Modeling Parameters[3]			Acceptance Criteria[3]				
			Plastic Rotation Angle, radians		Residual Strength Ratio	Plastic Rotation Angle, radians				
						Component Type				
						Primary			Secondary	
						Performance Level				
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.005	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ *d*/2			0.0	0.02	0.2	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > *d*/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ *d*/2			0.0	0.02	0.0	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > *d*/2			0.0	0.01	0.0	0.0	0.0	0.0	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

Table 6-7 ***Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Columns***

			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Component Type				
						Primary			Secondary	
			Plastic Rotation Angle, radians		Residual Strength Ratio	Performance Level				
Conditions			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	0.01	0.02	0.015	0.03
≤ 0.1	C	≥ 6	0.015	0.025	0.2	0.005	0.01	0.015	0.01	0.025
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.0	0.005	0.015	0.010	0.025
≥ 0.4	C	≥ 6	0.01	0.015	0.2	0.0	0.005	0.01	0.01	0.015
≤ 0.1	NC	≤ 3	0.01	0.015	0.2	0.005	0.005	0.01	0.005	0.015
≤ 0.1	NC	≥ 6	0.005	0.005	–	0.005	0.005	0.005	0.005	0.005
≥ 0.4	NC	≤ 3	0.005	0.005	–	0.0	0.0	0.005	0.0	0.005
≥ 0.4	NC	≥ 6	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0
ii. Columns controlled by shear[1,3]										
Hoop spacing ≤ $d/2$, or $\frac{P}{A_g f'_c} \leq 0.1$			0.0	0.015	0.2	0.0	0.0	0.0	0.01	0.015
Other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ $d/2$			0.01	0.02	0.4	1	1	1	0.01	0.02
Hoop spacing > $d/2$			0.0	0.01	0.2	1	1	1	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$[1,3]										
Conforming reinforcement over the entire length			0.015	0.025	0.02	0.0	0.005	0.001	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Table 6-8 ***Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beam-Column Joints***

			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Component Type				
						Primary			Secondary	
			Shear Angle, radians		Residual Strength Ratio	Performance Level				
Conditions			d	e	c	IO	LS	CP	LS	CP
i. Interior joints										
$\frac{P}{A_g f'_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]								
≤ 0.1	C	≤ 1.2	0.015	0.03	0.2	0.0	0.0	0.0	0.02	0.03
≤ 0.1	C	≥ 1.5	0.015	0.03	0.2	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≤ 1.2	0.015	0.025	0.2	0.0	0.0	0.0	0.015	0.025
≥ 0.4	C	≥ 1.5	0.015	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≤ 1.2	0.005	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≤ 1.2	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
ii. Other joints										
$\frac{P}{A_g f'_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]								
≤ 0.1	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≤ 0.1	NC	≤ 1.2	0.005	0.01	0.2	0.0	0.0	0.0	0.005	0.01
≤ 0.1	NC	≥ 1.5	0.005	0.01	0.2	0.0	0.0	0.0	0.005	0.01
≥ 0.4	NC	≤ 1.2	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0
≥ 0.4	NC	≥ 1.5	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0

Elaboración modelo analítico

- **Características mecánicas**
  - **Rigidez**
  - **Resistencia**
  - **Capacidad de deformación**
- **Rigidez diafragma**
- **Modelado tridimensional**
- **Fuerzas fuera del plano**

Rigidez:

Table 6-4 Effective Stiffness Values

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams—nonprestressed	$0.5E_cI_g$	$0.4E_cA_w$	—
Beams—prestressed	E_cI_g	$0.4E_cA_w$	—
Columns in compression	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns in tension	$0.5E_cI_g$	$0.4E_cA_w$	E_sA_s
Walls—uncracked (on inspection)	$0.8E_cI_g$	$0.4E_cA_w$	E_cA_g
Walls—cracked	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Flat Slabs—nonprestressed	See Section 6.5.4.2	$0.4E_cA_g$	—
Flat Slabs—prestressed	See Section 6.5.4.2	$0.4E_cA_g$	—

Note: I_g for T-beams may be taken as twice the value of I_g of the web alone, or may be based on the effective width as defined in Section 6.4.1.3.
For shear stiffness, the quantity $0.4E_c$ has been used to represent the shear modulus G.

Resistencia:

Table 5-2 Behavior Modes for Reinforced Concrete Wall Components.

Behavior Mode	Approach to calculate strength (use expected material values)	Approach to estimate displacement capacity	Ductility Category
A. Ductile flexural response	Conventional calculations per Section 5.3.5.	Good displacement capacity (e.g. 2% drift, or 8x yield displacement).	High ductility capacity
B. Flexure/ Diagonal tension	Moment strength per Section 5.3.5 initially governs strength.	Based on shear strength as a function of ductility. See Section 5.3.6.b.	Ductility capacity varies (Failure only occurs after some degree of flexural yielding and concrete degradation.)
C. Flexure/ Diagonal compression (web crushing)		Based on relationship of web crushing strength to drift per Oesterle et al (1983). See Section 5.3.6.c.	
D. Flexure/ Sliding shear		Shear friction approach per ACI 318, or recommendations of Paulay and Priestley (1992). See Section 5.3.6.d.	
E. Flexure/ Boundary-zone compression		Based on amount of ties required for moderate and high ductility levels, per Paulay and Priestley (1992). See Section 5.3.7.	
F. Flexure/Lap-splice slip		Based on lap strength as a function of ductility. See Section 5.3.8	
G. Flexure/Out-of-plane wall buckling		Based on wall thickness requirements for moderate and high ductility levels. See Section 5.3.9.	
H. Preemptive diagonal tension	Shear strength governs at low ductility levels, per Section 5.3.6.b.	No inelastic displacement capacity.	Little or no ductility capacity (Flexural reinforcement does not yield.)
I. Preemptive web crushing	May occur at shear stresses of $12\sqrt{f'_{ce}}$ – $15\sqrt{f'_{ce}}$. See Section 5.3.6.c.		
J. Preemptive sliding shear	Shear friction approach per ACI 318. See Section 5.3.6.d.		
K. Preemptive boundary zone compression	Applies only to unusually high axial loads, above the balance point. Moment strength calculation still governs.		
L. Preemptive lap-splice slip	Lap strength, per FEMA 273 and ATC-40, or approach of Priestly et al. (1996) governs. See Section 5.3.8.		
M. Global foundation rocking of wall	See FEMA 273 or ATC-40		Moderate to high ductility capacity
N. Foundation rocking of individual piers			

Resistencia:

5.3.2 Expected Strength and Material Properties

a. Expected Strength

The capacity of reinforced concrete components is calculated initially using expected strength values. Expected strength is defined in Section 6.4.2.2 of FEMA 273 and Section 9.5.4.1 of ATC-40 as "the mean maximum resistance expected over the range of deformations to which the component is likely to be subjected."

Expected component strength may be calculated according to the procedures of ACI 318 — or other procedures specified in this document — with a strength-reduction factor, ϕ, taken equal to 1.0. Expected material strength rather than specified minimum material strength is used in the calculations. Material strength values are discussed below.

Resistencia:

Table 6-1 Tensile and Yield Properties of Concrete Reinforcing Bars for Various Periods

		Structural[1]	Intermediate[1]	Hard[1]			
	Grade	**33**	**40**	**50**	**60**	**70**	**75**
Year[3]	Minimum Yield[2] (psi)	33,000	40,000	50,000	50,000	60,000	75,000
	Minimum Tensile[2] (psi)	55,000	70,000	80,000	90,000	80,000	100,000
1911-1959		x	x	x			
1959-1966		x	x	x	x		x
1966-1972			x	x	x		
1972-1974			x	x	x		
1974-1987			x	x	x	x	
1987-present			x	x	x	x	x

General Note: An entry "x" indicates the grade was available in those years.

Specific Notes: 1. The terms structural, intermediate, and hard became obsolete in 1968.
2. Actual yield and tensile strengths may exceed minimum values.
3. Until about 1920, a variety of proprietary reinforcing steels were used. Yield strengths are likely to be in the range from 33,000 psi to 55,000 psi, but higher values are possible Plain and twisted square bars were sometimes used between 1900 and 1949.

Resistencia:

Table 6-2 Tensile and Yield Properties of Concrete Reinforcing Bars for Various ASTM Specifications and Periods

				Structural[1]	Intermediate[1]	Hard[1]			
			Grade	**33**	**40**	**50**	**60**	**70**	**75**
			Minimum Yield[2] (psi)	33,000	40,000	50,000	50,000	60,000	75,000
ASTM	Steel Type	Year Range[3]	Minimum Tensile[2] (psi)	55,000	70,000	80,000	90,000	80,000	100,000
A15	Billet	1911-1966		x	x	x			
A16	Rail[4]	1913-1966				x			
A61	Rail[4]	1963-1966					x		
A160	Axle	1936-1964		x	x	x			
A160	Axle	1965-1966		x	x	x	x		
A408	Billet	1957-1966		x	x	x			
A431	Billet	1959-1966							x
A432	Billet	1959-1966					x		
A615	Billet	1968-1972			x		x		x
A615	Billet	1974-1986			x		x		
A615	Billet	1987-1997			x		x		x
A616	Rail[4]	1968-1997				x	x		
A617	Axle	1968-1997			x		x		
A706	Low-Alloy[5]	1974-1997						x	
A955	Stainless	1996-1997			x		x		x

General Note: An entry "x" indicates the grade was available in those years.

Specific Notes: 1. The terms structural, intermediate, and hard became obsolete in 1968.
2. Actual yield and tensile strengths may exceed minimum values.
3. Until about 1920, a variety of proprietary reinforcing steels were used. Yield strengths are likely to be in the range from 33,000 psi to 55,000 psi, but higher values are possible Plain and twisted square bars were sometimes used between 1900 and 1949.
4. Rail bars should be marked with the letter "R." Bars marked "s!" (ASTM 616) have supplementary requirements for bend tests.
5. ASTM steel is marked with the letter "W."

Resistencia:

Table 6-3 Compressive Strength of Structural Concrete (psi)[1]

Time Frame	Footings	Beams	Slabs	Columns	Walls
1900–1919	1000–2500	2000–3000	1500–3000	1500–3000	1000–2500
1920–1949	1500–3000	2000–3000	2000–3000	2000–4000	2000–3000
1950–1969	2500–3000	3000–4000	3000–4000	3000–6000	2500–4000
1970–Present	3000–4000	3000–5000	3000–5000	3000–10000[2]	3000–5000

1. Concrete strengths are likely to be highly variable within any given older structure.
2. Exceptional cases of very high strength concrete may be found.

Resistencia:

Table 5-2 Behavior Modes for Reinforced Concrete Wall Components.

Behavior Mode	Approach to calculate strength (use expected material values)	Approach to estimate displacement capacity	Ductility Category
A. Ductile flexural response	Conventional calculations per Section 5.3.5.	Good displacement capacity (e.g. 2% drift, or 8x yield displacement).	High ductility capacity
B. Flexure/ Diagonal tension		Based on shear strength as a function of ductility. See Section 5.3.6.b.	
C. Flexure/ Diagonal compression (web crushing)	Moment strength per Section 5.3.5 initially governs strength.	Based on relationship of web crushing strength to drift per Oesterle et al (1983). See Section 5.3.6.c.	Ductility capacity varies
D. Flexure/ Sliding shear		Shear friction approach per ACI 318, or recommendations of Paulay and Priestley (1992). See Section 5.3.6.d.	(Failure only occurs after some degree of flexural yielding and concrete degradation.)
E. Flexure/ Boundary-zone compression		Based on amount of ties required for moderate and high ductility levels, per Paulay and Priestley (1992). See Section 5.3.7.	
F. Flexure/Lap-splice slip		Based on lap strength as a function of ductility. See Section 5.3.8	
G. Flexure/Out-of-plane wall buckling		Based on wall thickness requirements for moderate and high ductility levels. See Section 5.3.9.	
H. Preemptive diagonal tension	Shear strength governs at low ductility levels, per Section 5.3.6.b.		
I. Preemptive web crushing	May occur at shear stresses of $12\sqrt{f'_{ce}}$ – $15\sqrt{f'_{ce}}$. See Section 5.3.6.c.		
J. Preemptive sliding shear	Shear friction approach per ACI 318. See Section 5.3.6.d.	No inelastic displacement capacity.	Little or no ductility capacity (Flexural reinforcement does not yield.)
K. Preemptive boundary zone compression	Applies only to unusually high axial loads, above the balance point. Moment strength calculation still governs.		
L. Preemptive lap-splice slip	Lap strength, per FEMA 273 and ATC-40, or approach of Priestly et al. (1996) governs. See Section 5.3.8.		
M. Global foundation rocking of wall	See FEMA 273 or ATC-40		Moderate to high ductility capacity
N. Foundation rocking of individual piers			

5.3.5 Moment Strength

The moment strength of a reinforced concrete component under flexure and possible axial loads is calculated according to conventional procedures, as defined in ACI-318, Section 10.2 (ACI, 1995), except that expected material strengths are used as discussed in Section 5.3.2 of this document. The moment strength accounts for all reinforcement that contributes to flexural strength. For example, the moment strength for a wall pier (component type RC1 or RC2) includes all well-anchored vertical bars at the section of interest, not just those in the wall boundaries. The axial load present on the wall component is taken into account in the calculation of moment strength.

a. Uncertainties or discrepancies in strength

Typically, there should be little uncertainty in the calculation of moment strength for a reinforced concrete component if reinforcement sizes, layout, and the steel and concrete material strengths have been established. The possible range of axial load on the component must also be considered.

Y finalmente, la capacidad deformación…

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

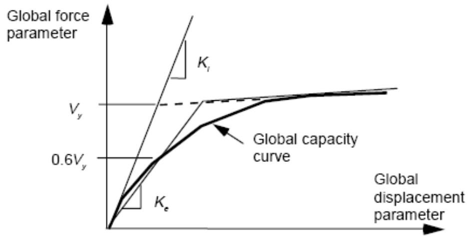
			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Component Type				
						Primary			Secondary	
			Plastic Rotation Angle, radians		Residual Strength Ratio	Performance Level				
Conditions			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.005	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ *d*/2			0.0	0.02	0.2	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > *d*/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ *d*/2			0.0	0.02	0.0	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > *d*/2			0.0	0.01	0.0	0.0	0.0	0.0	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

4.4.4 Global Displacement Demand

4.4.4.1 Displacement Coefficient Method

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \quad (4\text{-}1)$$

where T_i is the elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis, K_i is the elastic lateral stiffness of the building in the direction under consideration (refer to Figure 4-9), and K_e is the effective lateral stiffness of the building in the direction under consideration (refer to Figure 4-9). As described



The target displacement, δ_t, is calculated as:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2}$$

C_0 = Modification factor to relate spectral displacement and likely building roof displacement

Table 3-2 Values for Modification Factor C_0

Number of Stories	Modification Factor[1]
1	1.0
2	1.2
3	1.3
5	1.4
10+	1.5

1. Linear interpolation should be used to calculate intermediate values.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2}$$

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response

= 1.0 for $T_e \geq T_0$

= $[1.0 + (R-1)T_0/T_e]/R$ for $T_e < T_0$

Values for C_1 need not exceed those values given in Section 3.3.1.3.
In no case may C_1 be taken as less than 1.0.

T_0 = Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum. (See Sections 2.6.1.5 and 2.6.2.1.)

R = Ratio of elastic strength demand to calculated yield strength coefficient. See below for additional information.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2}$$

C_2 = Modification factor to represent the effect of hysteresis shape on the maximum displacement response. Values for C_2 are established in Section 3.3.1.3.

Table 3-1 Values for Modification Factor C_2

	$T = 0.1$ **second**		$T \geq T_0$ **second**	
Performance Level	**Framing Type 1**[1]	**Framing Type 2**[2]	**Framing Type 1**[1]	**Framing Type 2**[2]
Immediate Occupancy	1.0	1.0	1.0	1.0
Life Safety	1.3	1.0	1.1	1.0
Collapse Prevention	1.5	1.0	1.2	1.0

1. Structures in which more than 30% of the story shear at any level is resisted by components or elements whose strength and stiffness may deteriorate during the design earthquake. Such elements and components include: ordinary moment-resisting frames, concentrically-braced frames, frames with partially-restrained connections, tension-only braced frames, unreinforced masonry walls, shear-critical walls and piers, or any combination of the above.
2. All frames not assigned to Framing Type 1.

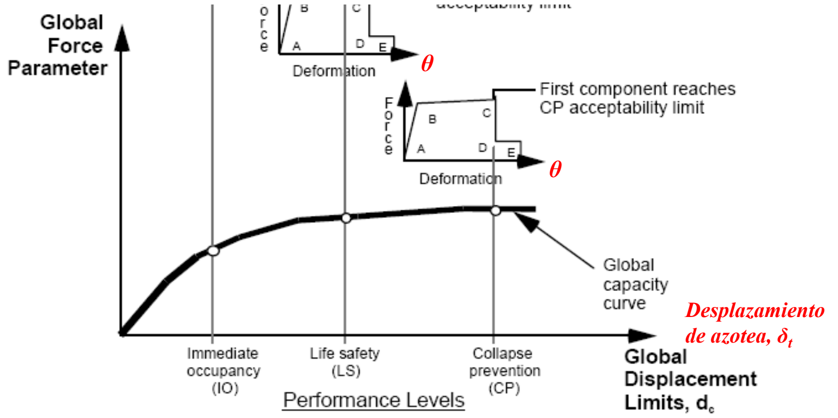
$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2}$$

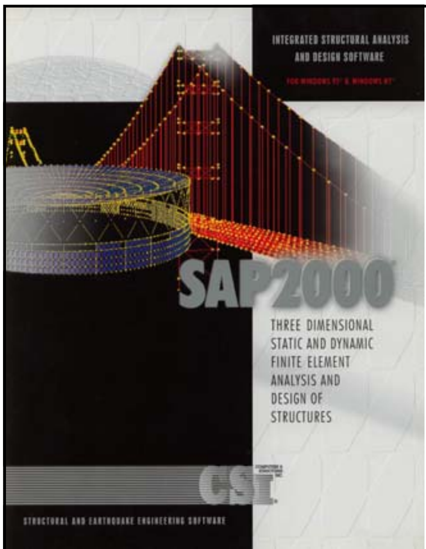
C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects. For buildings with positive post-yield stiffness, C_3 shall be set equal to 1.0. For buildings with negative post-yield stiffness, values of C_3 shall be calculated using Equation 3-13. Values for C_3 need not exceed the values set forth in Section 3.3.1.3.

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \qquad (3\text{-}13)$$

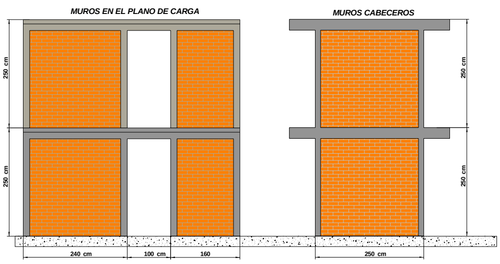
La revisión de la capacidad de desplazamiento lateral de la estructura debe hacerse para todos los objetivos de diseño relevantes. La condición crítica que surja a partir de esto define el estado de la estructura

En función de los valores de δ_t asociados a sismos de diferente intensidad, es posible evaluar si el desempeño sísmico de la edificación. En caso que no, la edificación debe rehabilitarse. Note que es posible usar el mismo esquema de evaluación para evaluar la efectividad del esquema de rehabilitación.

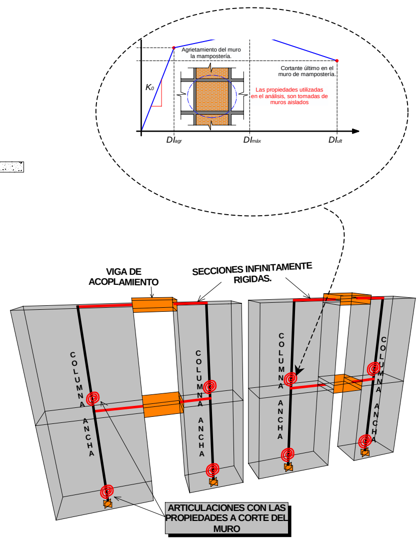




Análisis no lineal de las estructuras de mampostería.

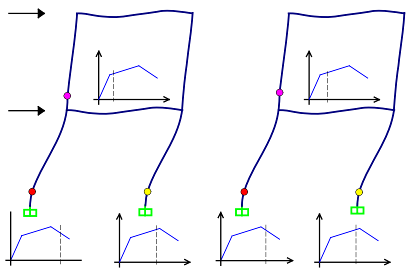


Modelo 3D (Alcocer, 1993)

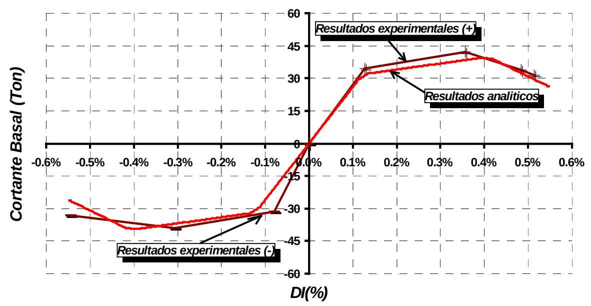


El modelo de columna ancha da resultados razonables en el modelado analítico de las estructuras de mampostería. Sin embargo, las propiedades de las columnas que modelan los muros deben contemplar el comportamiento no lineal de la mampostería.

Análisis no lineal de las estructuras de mampostería.

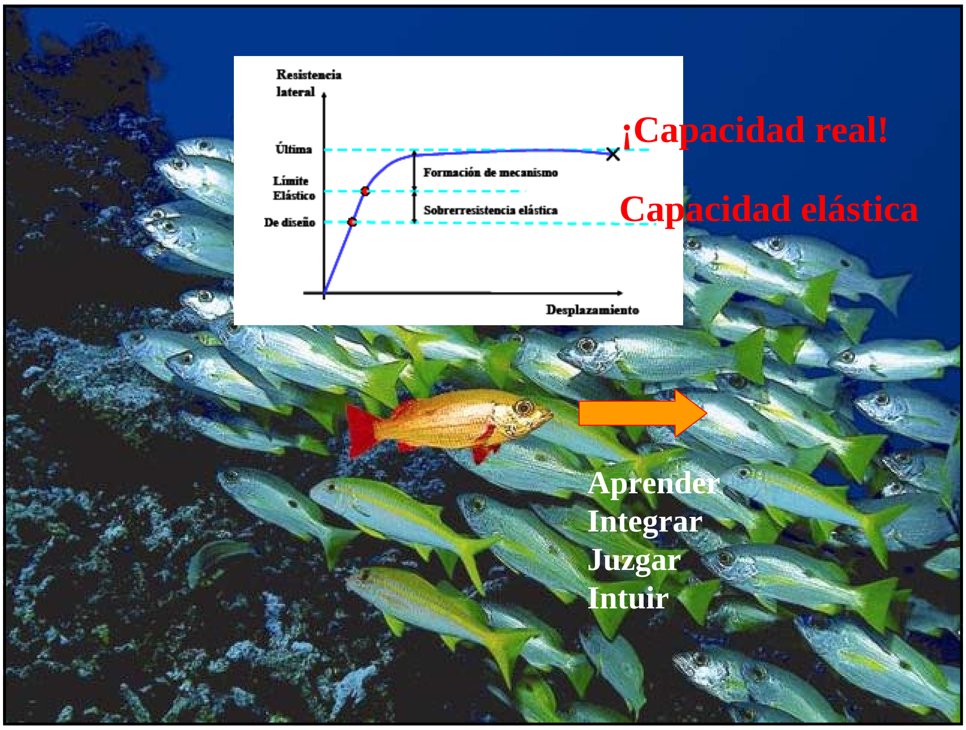


El análisis debe contemplar el comportamiento local y global de la estructura.



¿Porqué hacer una evaluación por desplazamientos?


¡¡¡Mejor no, por que es mas difícil!!!
Inercia


Resistencia lateral
Última
Límite Elástico
De diseño
Formación de mecanismo
Sobrerresistencia elástica
Desplazamiento
¡Capacidad real!
Capacidad elástica
Aprender
Integrar
Juzgar
Intuir


¡Por que es necesario hacer ingeniería que resulte en una solución adecuada desde puntos de vista técnico y económico!