POST TENSION FLAT SLABS—EARTHQUAKE ENGINEERING & MISC ASPECTS IN HIGH SEISMIC ZONES IN THE INDIA ENVIRONMENT

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ABSTRACT

Design related information to Post Tension (PT) flat slab is sketchy in the current Indian codes of practice. The practice of PT flat slab design for gravity loads is basically done as a complete design and build job by the PT manufacturer whereas earthquake provisions are rarely addressed. Though the structural stability certificate is signed by the structural engineer, the responsibility of design remains blurred. Design and construction faults are common and there is an alarming need of awareness, both among designers as well as contractors. The paper tries to find solutions to this potentially catastrophic approach being followed in the industry today by clarifying some key provisions of foreign codes such as ACI-318 & AS3600 and combining them with broad guidelines given in Indian codes and citing some examples of failures. Also a new methodology of seismic design of PT flat slabs for Indian situations is suggested.

Key Words:
Post-Tension, flat slab, 1893, seismic design, earthquake, 13920, equivalent frame, dual system, deformation compatibility.

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INTRODUCTION

Even though flat slab construction is not a new concept in the Indian environment, its widespread use especially with Post Tension (PT) application is fairly recent especially in high earthquake zones of the country. And since no comprehensive Indian standards exist on the subject, Structural Engineers working in high seismic zones have been eyeing this type of design and construction with skepticism.

Modern concrete construction in high seismic zones of India has traditionally been done using Special Moment Resisting Frames (SMRF)—ref IS 1893-2002, with or without shear walls. The columns are designed to be stronger than the beams. Ductile detailing provisions of IS (13920-1993) ensures this. After the Bhuj earthquake of 2001 the compliance with these standards has become in vogue. The code allows the response reduction factor (R) to be as high as 5 for SMRF to 4 for buildings with shear walls alone taking base shear more than 75%, and 5 for "dual systems" ie buildings in which both shear walls and SMRF exist while the latter take at least 25 % of the base shear.

As there are no beams, flat slab structures do not strictly fall in the category of frames—at least as per all existing Indian standards. Hence providing shear walls is mandatory in buildings to take the entire lateral seismic force. However there is an argument that the basis of flat slab design assumes an "equivalent frame" in orthogonal directions of the building so it may be considered as a frame for earthquake purposes as well. To address this view point it is useful to look at provisions of some codes like ACI-318-05, ASCE-41-06, that are based on more recent research on the subject.

Subsequently a design methodology will be introduced that is based on broad principles laid out by the Indian code, recent research from literature and experience in the design office.

ACI-318 PROVISIONS FOR FLAT SLAB

ACI considers design of flat slab from two distinct view points. One is the flat slab and column structure designed as an "Intermediate Frame". The second is the flat slabs with columns that do not participate in resisting seismic forces. The structure relies on a defined lateral force resisting system such as shear walls to take all lateral forces.

However there is a limitation on considering flat slab & column structure as an Intermediate Frame. These types of structures are not allowed in high seismic areas. Further there are some strict guidelines as to the amounts of steel rebar (usually non-PT) required in a typical bay to take these forces.

Intermediate frame provisions include requiring a minimum amount of rebar passing through the column cage along with the PT tendons. In fact the emphasis on this point is so strong that ASCE-41-03 "Seismic evaluation of Existing Buildings" states that a building cannot be considered as "Immediate Occupancy" or "Life Safety" if it has no continuous bottom steel. It also states that the ductile rotation capacity & Residual Strength of a connection is zero if no bottom bars & PT TENDON are passing through the COLUMN CAGE

Practically the authors found that practice in the US is that the use of intermediate frame concept is sparse and even in areas such as the lowest seismic zones (like New York), shear walls are used.

The other approach of not considering the flat slab to be part of the lateral force resisting system is more popular and strongly endorsed by the authors. In this approach the storey drift is related to the state of punching shear stress in the column-slab connection (that is un-reinforced for punching).

Note that ACI states that the column moment is transferred to the slab via bending and eccentric punching shear. Further it requires the total punching shear stress (direct plus eccentric) to include the effect of moment at the slab/column joint.

For high punching shear stresses (with no punching shear reinforcement) lesser storey drift is allowed—see Fig 1. Hence anyone designing using this approach would need to make the shear walls stiffer so as to lower storey drift and in turn shear reinforcement is not required in the joint.

If punching shear reinforcement is unavoidable then it may be provided in the form of a "shear stud rail" system see Fig 2. This is favoured over the conventional stirrup reinforcement due to its ineffectiveness in thin slabs. Conventional stirrups should not be provided in slabs thinner than 250 mm thick.

INDIAN SCENARIO:

Translated to the Indian context it would appear fair to say that intermediate frames, if at all used should be confined to Zone II and perhaps in the exceptional case to Zone III.

Following is an excerpt from IS1893-2002, Clause 7.11.2 "Deformation Capability"—

- "For buildings located in Zones IV & V, it shall be ensured that structural components, that are not part of the seismic force resisting system in the direction under consideration do not loose their load-carrying capacity under the induced moments resulting from storey deformations equal **to** R times the storey displacements calculated as per 7.11.1…."

With the following footnotes:

- "For instance a flat-slab building in which lateral load resistance is provided by shear walls. Since the *lateral load resistance* of the *slab-column system is small*, these are often designed only for gravity loads, while all the seismic force is resisted by shear walls.
- Even though the slabs & columns are not required to share the lateral forces, these deform with the rest of the structure under seismic force.
- The concern is that under such deformations, the slab-column system should not loose its vertical load capacity."

This literally implies that when the flat slab components (columns, slabs & their joints) that are not considered part of the lateral force resisting system, R should be taken as 1 for the purpose of design of these components.

This clearly almost agrees with the ACI approach (which prohibits use of Intermediate Frames in high seismic risk zones. Other than the statements mentioned above there is little else found in any Indian standard and one is left looking at "specialist literature" such as ACI-318 which is unequivocally one of the most accepted standards for earthquake resistant flat slab construction.

LIMITATIONS IN DESIGN & CONSTRUCTION IN THE INDIAN SCENARIO

Currently a limitation faced by Indian designers in the construction of flat slabs is non-availability of shear stud rails which is hugely popular across countries such as US, Australia, UK & Europe in general. As the ACI prohibits the use of shear stirrups in slabs thinner than 150 mm thick and discourages their use in slabs thinner than 250 mm, one is left with little option than to provide drops. People designing PT flat slabs will vouch that it is aesthetically an undesirable option and is a non-seller with most clients.

The other limitation faced by designers is availability of experienced personnel. The design of PT flat slabs for gravity loads is largely done by PT manufacturer. With most projects being "design-built" the project structural engineer has a reduced role but greater responsibility—especially on parts he has little control. Many designers tend to pass on the responsibility to those employed by the PT manufacturers. Then reviewing PT designs is just a rubber stamping chore. However where everyone tends to loose out is safety of the structure.

COMMON DESIGN/CONSTRUCTION DEFECTS IN PT FLAT SLABS

The authors have observed common design defects/failures in construction of PT flat slabs. Some of these are presented here for discussion with the hope that at least these defects are recognized by people associated with the design/construction of flat slabs.

It is desirable to separate the shear walls or other such stiff elements from PT slabs during the pre-stressing operation. This is required as the resistance offered by such elements is high and substantial PT force is lost in them. This makes the slabs dangerous as they do not contain the PT force the designer has intended.

Fig 11 shows a slab under construction. Note that this construction is below ground level and there are RC basement/shear walls visible in the picture. There is no special detail that temporarily separates the two at the PT stage. Authors recommend that PT should not be done in slabs at or below ground level due to presence of basement/retaining walls and their integration with the slabs—as is also the general practice in US, Europe and Australia.

As already discussed above, it is desirable to have the PT tendons run through the column cage. A PT slab construction (Fig 3) shows ducts going on either side but not through the cage. Also note that if individual strands were used as is the practice in US (Fig 4) then it would be feasible for them to be taken through the column cage. With wide ducts as is common practice in India, it would be difficult to accommodate the two together. Therefore authors suggest that individual 7 wire strands be used instead of the wide duct (Fig 4).

Further as per common practice by PT designers, long term cracked deflection criterion given in the code is usually not being checked. There have been several reported functional failures on account of this criterion. See Fig 5, 6 & 7 that show the difference in calculated deflection due to elastic, long term uncracked and long term cracked deflections respectively. Note the same spot shows 3.88 mm, 9.24 mm and 15.3 mm deflection respectively. RAM Concept software was used in this calculation. The program uses Equivalent Curvature Ratio principle for cracked areas of the slab and using weights for those areas that are cracked versus uncracked. Despite minor inaccuracies that may exist this calculation reveals the order of difference that can exist when long term cracked deflections are not included.

In order to prevent a progressive collapse in flat slabs, the Australian code (AS3600) has mentioned that there should be bottom steel at the slab column connection (see Fig 8). This figure shows the quantity of steel required at that place. A quick run through the numbers reveals some astonishing amount of steel required. As per observation the top bars are ineffective during a punching shear failure event. The bottom bars begin to take the force in the form of a centenary.

See Figures 9 & 10 for pictures (note the source is unknown, but the picture was downloaded from Structural Engineer's forum of India web-site) of a collapsed building that used flat slabs. Note there are some bottom bars visible in the picture. The news reports stated that there was approximately one hour time from the time excessive deflections were noted on one slab to the time that the whole structure collapsed. It is the authors' conviction that this time lag (which saved lives) was due to the bottom steel provided— albeit inadequate amount.

REASONING OF PROPOSED APPROACH

In order to understand the proposed approach we first understand the concept of the response reduction factor (R). If no ductility exists in an element it must be designed for R=1. For example if the shear wall is to be designed for R=4 implies that a hinge will be formed in it at the reduced force level, but it is likely to continue to deform while offering constant resistance till some point and then eventually degrade to a much lower resistance level. Fig 12 (ref ASCE 41-06) shows this idealized hinge behaviour of not only a wall but any element that deforms.

The logic used by IS 1893-2002, in suggesting R=1 for members & joints not intended to be part of the lateral force resisting system is not difficult to see. Even as the shear walls get a "reprieve" of being designed to R=4, the thought process for slabs & columns is <u>Deformation Compatibility</u>**,** since the deformations these members must experience are at actual levels of earthquake.

However by extending this logic to all elements they do become quite uneconomical. In some sample cases it was found that an incredible increase of steel was required in the PT slab when column moments at R=1 was transferred to the slab.

So to find a middle path, one can look at the rotation allowed in the columns after the hinges have been formed. If there is sufficient rotation allowed till R=1 then the intent of the code is achieved. This concept is reflected in the retrofitting code--ASCE-41.

This code shows two relevant analysis procedures that may be followed in evaluating structures. One is the Non-linear Static Procedure (NSP)—commonly called pushover analysis, and the second is the Linear Response Spectrum Analysis (LRSA). LRSA is commonly used in the Indian context so the proposed design approach will use this procedure.

Even while performing an LRSA procedure ASCE-41 helps achieve the objective of a non-linear analysis, which is more accurate than the LRSA, by classifying "performance based design" with performance levels, i.e. Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). It is stated elsewhere in this code that for IO performance levels the component being designed is just into the inelastic range (minor cracking). From IO to LS there is no degradation of strength if it is a ductile system. But there is a notable drop in strength between LS to CP states. However the focus here will be on the strength drop between IO and LS limit states—if any.

Further instead of using one "R" value for the analysis of the whole building with a single R=1, various "R" values are assigned to components like beams, columns, slabs or joints. In this code the term "R" factor for individual elements is referred to as "m" factors. These "m" factors have relation with the ductility of the individual element rather than the whole building.

Tables 1 and 2 (Tables 6-8 & 6-12 in ASCE-41) show the rotation of the columns from NSP & "m" factors from the LRSA points of view respectively. Note that the "m" factors increase as the performance levels reduce from IO to LS & then to CP. The change of rotation angle (or "m" factor) may be interpreted as the multiple of the IO state to LS or CP. For example if the rotation at IO state is .01 radians and that at LS is .03 radians, then the column hinge can rotate to three times the rotation at the IO state. Likewise if the "m" factor is one for IO state & three for LS state, then the a similar argument would hold for LRSA procedure . Tables 3 and 4 (Tables 6-14 & 6-15 respectively of ASCE-41) show the same thing but for slab column joints.

ASCE-41 also classifies the components between primary and secondary. Members that are not part of the primary lateral force resisting system would be termed secondary. As an example when columns are not part of the lateral force resisting system, they would be secondary, but in Zone 2 if they are part of an "Intermediate Frame" they would be termed primary.

Note that for slab-column joints of PT slabs, in Table 4 for medium loaded slabs ($Vg/Vo \geq 0.6$) the "m" factors remain as 1 for all states from IO to CP for primary members but vary from 1 to 2 & then 2 to 2.25 for secondary elements for same limit states. For this case of a secondary component R can be taken as 2 for connection design (i.e. considering only IO to CP—without degradation of strength). But for a primary system the same component would be designed for R=1.

Thus most structures must be analyzed using several models—with as many values of R as necessary—depending on number of "m" factors, as mentioned above.

In order to ensure a ductile mode of failure, punching shear overstress must be prevented at all costs. Just as the "strong column-weak beam" concept protects catastrophic failures in SMRF, strong slab-column punching connection & weak column prevents sudden & progressive collapse in flat slabs (ie punching shear failure is prevented). So the order of strength is first (strongest) punching shear, then column & finally slab.

The argument in ACI-318 is that if the load to (unreinforced) punching shear stress ratio is high (above 0.6), the maximum allowable drift is 0.005 (based on cracked section analysis). Even though the maximum allowable drift in the IS-1893-2002 is 0.004, it is based on gross section analysis (with R=4). Note that ASCE-41 for cracked section, suggests an "EI" cracked to gross ratio of 0.3 to 1 depending level of axial stress for beams & columns.

DESIGN METHODOLOGY

In the Indian scenario for a PT flat slab building, one analysis would be for R=4, with columns moment released top & bottom The results of this analysis are used for designing shear walls & foundations—for the primary lateral force resisting system taking 100% of lateral forces.

Second analysis for column design with R=3 with uncracked sections for all elements. These moments would also be transferred to the slab for its design. This analysis is recommended with uncracked sections as this is really the intent if the Indian codes.

Third, for slab-column connection, the designer has an option to follow the ACI-318 (Fig 1) with cracked section & R=3 but columns are monolithic. The drift is checked from the curve & seen whether shear reinforcing is required. Alternatively analysis of slab column connection with R=2 & cracked section. If it is desired not to provide shear reinforcement the combination of limiting storey drift and increasing slab/drop thickness etc. should be done so the joint is compliant. Therefore all ductility & progressive collapse provisions of the ACI-318 & ASCE-41 should be followed. Here columns are considered cracked as design process is relying on ACI-318, which this requirement and no equivalent is found in the Indian Codes. ***If R=2 is adopted, the slab-column connection should also be checked so it is not designed for less than the residual moment capacity of the column (at the R value for which the column has been designed).***

See Table 5 for tabular presentation of these recommendations.

It may be noted that this represents a case with moderately stressed columns and connections & ductile provisions of ACI included in design. With heavily stressed conditions the user is referred to ASCE-41 for more restrictive "R" or "m" factors. Also accompanying footnotes & text of this reference, ACI-318 & AS3600 should be followed for detailing.

SUMMARY

The following are the main points highlighted in this paper:

1. An independent lateral force resisting system (such as shear walls) should be provided for PT Flat slab structures. For low seismic areas only, column & flat slabs, when designed as per rules of "Intermediate Frames", may be used to resist seismic forces—though the authors do not recommend this option.

2. A design methodology has been proposed that requires different levels of response reduction for various elements depending on conformance with detailing and consequently their level of ductility.

3. Minimum bottom steel should be provided in slab-column connection to prevent sudden collapse.

4. Separation between shear resisting elements & PT slab should be ensured prior to PT operation so stress is not lost to the shear resisting elements.

5. Some PT tendons should run through the column cage in both directions for better seismic performance.

6. It is important to check long term cracked deflections in PT flat slab to avoid functional failures (service limit state).

CONCLUSIONS & RECOMMENDATIONS

Design and construction of PT flat slabs requires significant improvement in India. Indian Codes of practice also need to be upgraded keeping in line with the state-of-the-art, some of which has been highlighted in this paper.

A design methodology for PT flat slab design has been proposed in this paper for the Indian environment based on current state of practice. Buildings should be analysed with various levels of "R" reflecting a design level for each type of structural element. Alternatively one analysis with R of 1 may be done and various "m" factors may be applied to the various elements based their ductility when designing them. Some examples of how to achieve this for some cases has been shown in the main text.

This paper would not be complete without stating that Punching shear failure should be avoided at all costs and measures should be built in to prevent progressive collapse. Also design of PT flat slabs should be brought into the ambit of the structural design consultant and not stay with the contractor.

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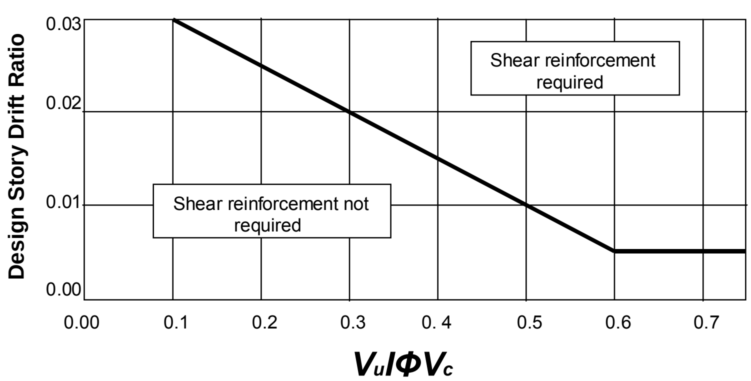

Fig 1: Storey drift ratio (based on cracked section) Vs gravity shear ratio

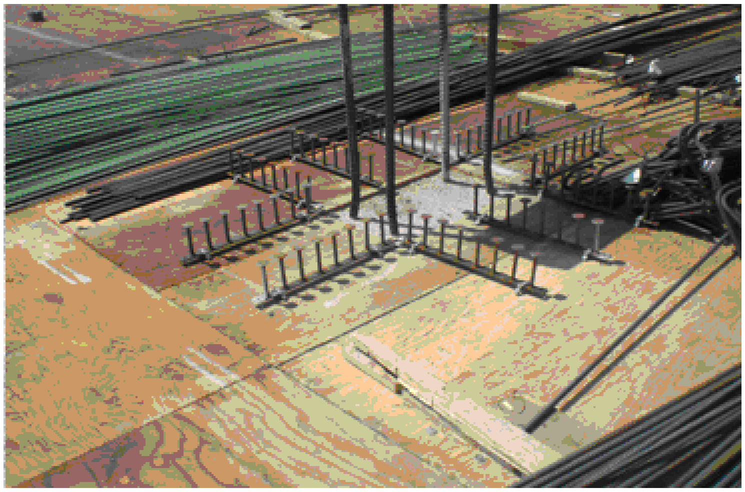

Fig 2: Shear Stud Rails Placed Around Column

Fig 3. Indian project—Tendons Not Going Through Column Cage

Fig 4. Typical Column-Slab Joint—USA East Coast Construction

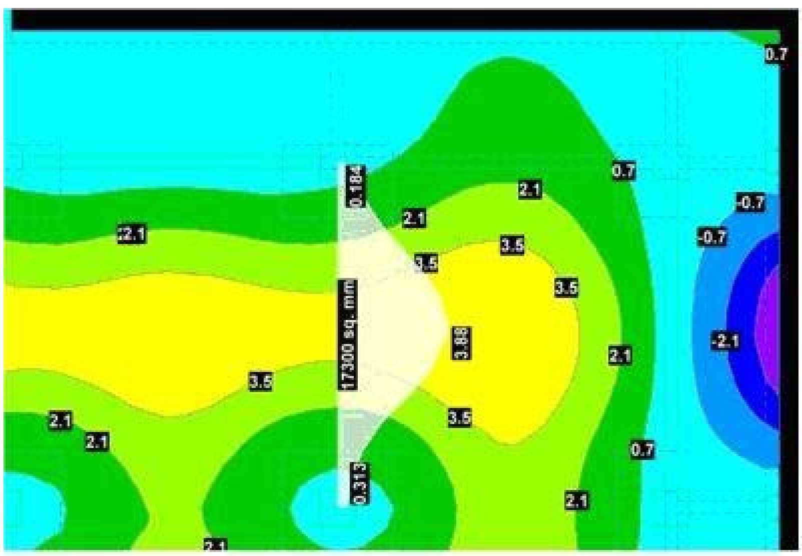

Fig 5: Short Term Uncracked Deflection

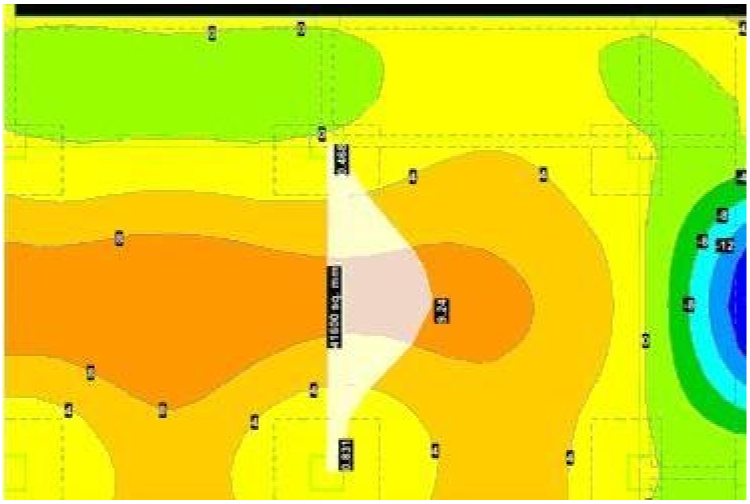

Fig 6. Long Term Uncracked Deflection

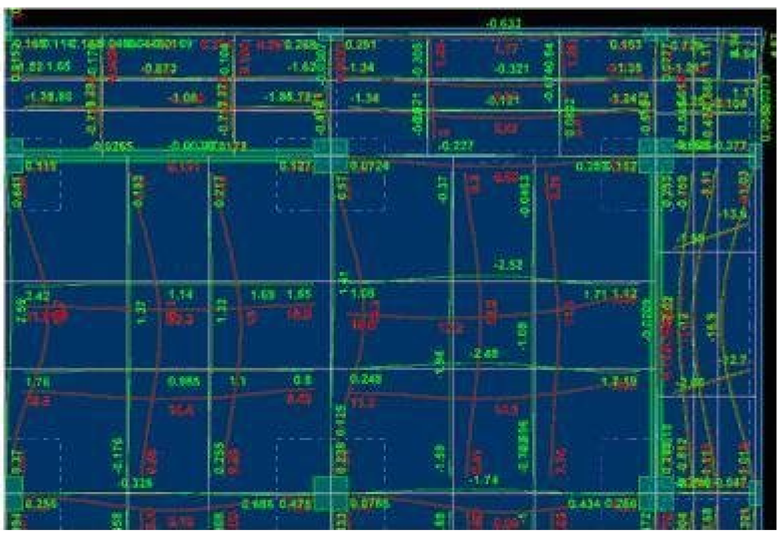

Fig 7: Long Term Cracked Deflection

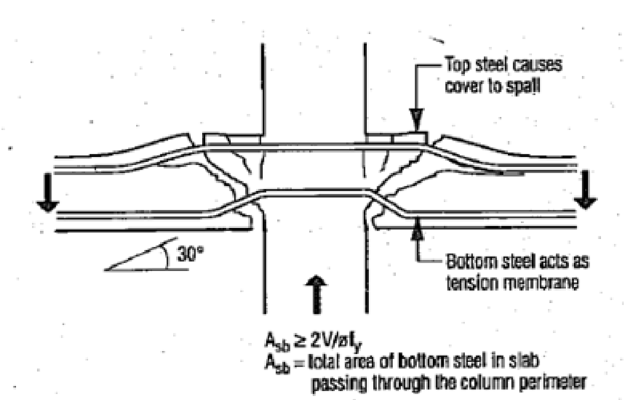

Fig 8: Tensile Membrane (Bottom) Steel at Column-Slab Intersection

Fig 9: Flat slab Bangalore building progressive collapse-2008

Fig 10: Flat Slab Bangalore Building Progressive Collapse—2008

Fig 11: Shear Walls Being Cast Monolithically with PT Slabs!!!

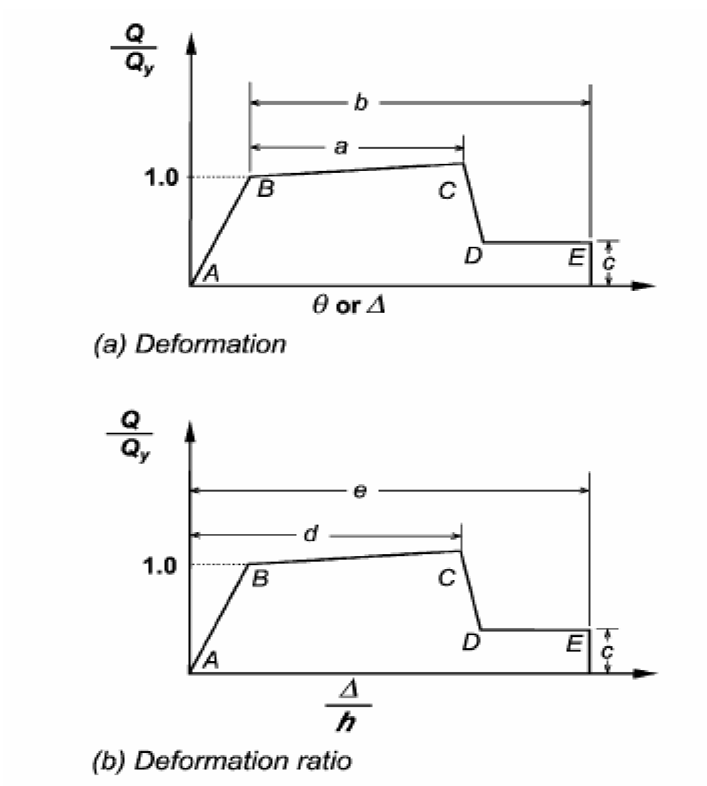

Fig 12: Generalized Force-Deformation Relations for Concrete Elements or Components

Table 1 Numerical Acceptance Citeria for Nonlinear Procedures
Reinforced Concrete Columns

		Acceptance Criteria				
		Plastic Rotation Angle, Radians				
		Performance Level				
			Component Type			
		IO	Primary		Secondary	
			LS	CP	LS	CP
$P/A_gf'c$	$\rho=As/b_ws$					
≤0.1	≥0.006	0.005	0.026	0.035	0.045	0.060
≥0.6	≥0.006	0.003	0.008	0.009	0.009	0.010
≤0.1	=0.002	0.005	0.020	0.027	0.027	0.034
≥0.6	=0.002	0.002	0.003	0.004	0.004	0.005

Note: Part of Table 6-8 extracted from ASCE-41-2006

Table 2 Numerical Acceptance Citeria for Linear Procedures
Reinforced Concrete Columns

		m-factors				
		Performance Level				
			Component Type			
			Primary		Secondary	
		IO	LS	CP	LS	CP
$P/A_gf'c$	$\rho=As/b_ws$					
≤0.1	≥0.006	2.0	2.5	3.0	4.0	5.0
≥0.6	≥0.006	1.3	1.8	1.9	1.9	2.0
≤0.1	=0.002	2.0	2.0	2.6	2.6	3.0
≥0.6	=0.002	1.1	1.1	1.2	1.2	1.4

Note: Part of Table 6-12 extracted from ASCE-41-2006

Table 3 Numerical Acceptance Citeria for Nonlinear Procedures
Two-way Slabs and Slab-Column Connections

		Acceptance Criteria				
		Plastic Rotation Angle, Radians				
		Performance Level				
			Component Type			
			Primary		Secondary	
		IO	LS	CP	LS	CP
Vg/Vo	Continuity Reinforcement					
0	Yes	0.01	0.026	0.035	0.035	0.050
0.2	Yes	0.01	0.023	0.03	0.03	0.04
0.4	Yes	0	0.015	0.02	0.02	0.03
≥0.6	Yes	0	0	0	0	0.020

Note: Part of Table 6-14 extracted from ASCE-41-2006

Table 4 Numerical Acceptance Citeria for Linear Procedures
Two-way Slabs and Slab-Column Connections

		m-factors				
		Performance Level				
			Component Type			
			Primary		Secondary	
		IO	LS	CP	LS	CP
Vg/Vo	Continuity Reinforcement					
0	Yes	2	2.75	3.5	3.5	4.5
0.2	Yes	1.5	2.5	3	3	3.75
0.4	Yes	1	2	2.25	2.25	3
≥0.6	Yes	1	1	1	1	2.25

Note: Part of Table 6-15 extracted from ASCE-41-2006

Table 5: Design Methodology For PT-Flat Slab Buildings in India

S. No	Component	R value	Column Joints Moment Released	Cracked/ uncracked sections	Shear Reinf reqd? (ACI 21.11.5)
1	Shear walls & foundations	4	Yes	Uncracked	N/A
2	Columns & Slabs	3	No	Uncracked	N/A
3a*	Column-slab Connection	3	No	Cracked	No
3b*	Column-slab Connection	2	No	Cracked	Yes

* Alternative approaches--Check 3a first--if non-compliant, adopt 3b