Basic Concepts

It is highly recommended to see - *Basic Concepts* before starting this topic.

What are stiffness modifiers?

Stiffness modifiers in ETABS are the factors to increase or decrease some properties of the cross section for example area, inertia, torsional constant etc.

Generally they are used to reduce stiffness of concrete sections to model for cracked behavior of concrete. They are only applied to concrete members because it cracks under loading.

Background

Design of sections is carried out based on the forces calculated from analysis of the structure. These forces depend upon stiffness of the members. ***Stiffness is the ability to attract moment, shear, axial force etc. Stiffer an element, more force it attracts and more reinforcement we design for.*** In a building some elements are more stiff, and others are less stiff. So they attract different amounts of forces depending upon their stiffness. Applied loading on a building produces internal forces. These internal forces likes flexure, shear, torsion and axial forces result in compression or tension in concrete fibers. Concrete is strong in compression but it is only good in tension as little as about 10% of its compressive strength.

At this limit, concrete cracks, reduces in area and stiffness. It is no longer available to resist tensile actions. As the stiffness reduces so does the moment attracting ability. Some of the moment which was present at this section (for example at beams) goes to other areas which are not yet cracked (for example columns). This reshuffling of the stiffness in the whole structure leads to redistribution of moments. So these un cracked regions (for example columns) have to be designed for more moment than what they actually received before moment redistribution. This phenomenon is called redistribution of moments.

Those regions which were un cracked and received extra moments from cracked regions will also crack once the concrete in that region reaches its tensile capacity limit. So this cycle of moment redistribution continues until all the members have been cracked. Steel reinforcement which sits idle before this stage now starts taking these redistributed moments.

Why we need to change stiffness, what is gross/cracked section analysis and are these modifiers for service or ultimate design?

Cracking will affect the stiffness of the structure which in result will affect the deflection and forces. No one knows how real the exact cracking would be, what would be its extent and how different the load distribution would be after that.

Since there is cracking that occurs in the section there is a variable moment of inertia. In regions of cracking there is a reduced moment of inertia and in areas without cracking the moment of inertia is much larger. Also, as the load increases, so does the cracking. The moment of inertia changes with load. This moment of inertia that changes with load makes our deflection equations non-linear with respect to loading and we cannot use superposition to determine load combinations.

A non-linear analysis shall be required to predict extent of cracking. For linear analysis it is necessary to have some reasonable assumptions about stiffness of the members.

Changing the relative stiffness of members of a structure so the intent is to have a good design by limiting its deflection, sway and cracking is a serviceability issue, that means we change stiffness, increase here and decrease there and change the analysis results to adjust our structure to control large deflections and cracks.

We may wish not to change the stiffness and just design on original stiffness of the members. That design may show large cracks and deflections and may impair its service use to the occupants but still it would be within the strength design limits. Analysis can be good or bad, more cracked or less cracked depending on our selection of member stiffnesses but the results obtained will be used to design the members.

Gross or cracked section means gross or cracked inertia of the section. Inertia has the same meaning as that of stiffness because E is held constant in all this discussion. As the stiffness is a function of E and I. For RC members, we can assume a constant E, If deformations stay within the elastic region (the normal assumption for most materials). The effective moment of inertia, I_e, is a moment of inertia that, when used with deflection equations developed for prismatic members, will yield approximately the same result as a more rigorous analysis that considers variable moment of inertia. This effective moment inertia will always be between the two extremes of the gross moment of inertia and the cracked moment of inertia, both of which have been discussed above.

You need reduced stiffness for 3 purposes:-

1) Vertical Deflections

2) Horizontal Deflections

3) Slenderness

The ACI 318 Building Code Requirements for Structural Concrete uses Limit state design.

Limit state design (LSD) *refers to a design method used in structural engineering. A limit state is a* ***CONDITION*** *of a structure beyond which it no longer fulfills the relevant* ***DESIGN CRITERIA****. The condition may refer to a degree of loading or other actions on the structure, while the criteria refer to structural integrity, fitness for use, durability or other design requirements.*

There are two categories of limit states: strength and serviceability.

http://en.wikipedia.org/wiki/Limit_state_design

What values to use?

Let's see what chapter 8 of ACI 318-08 "Analysis and Design - General Considerations" says about stiffness:-

ACI 8.7.1 *— Use of any set of reasonable assumptions shall be permitted for computing relative flexural and torsional stiffnesses of columns, walls, floors, and roof systems. The assumptions adopted shall be consistent throughout analysis.*

Ideally, the member stiffnesses ***EcI*** *and* ***GJ*** *should reflect the degree of cracking and inelastic action that has occurred along each member before yielding. However, the complexities involved in selecting different stiffnesses for all members of a frame would make frame analyses inefficient in design offices. Simpler assumptions are required to define flexural and torsional stiffnesses.*

ACI R8.8.1 *— The selection of appropriate effective stiffness values depends on the intended performance of the structure.* which means there are different assumptions of stiffness for different types of analysis. For example braced frames, sway frames, lateral analysis etc. We will explore them in detail.

Braced Frames

ACI R8.7.1 *— For braced frames, relative values of stiffness are important. Two usual assumptions are to use gross* ***EcI*** *values for all members or, to use half the gross* ***EcI*** *of the beam stem for beams and the gross* ***EcI*** *for the columns.*

Unbraced Frames

ACI R8.7.1 *— For frames that are free to sway, a realistic estimate of* ***EcI*** *is desirable and should be used if second-order analyses are carried out. Guidance for the choice of* ***EcI*** *for this case is given in R10.10.4.*

Torsional Stiffness

ACI R8.7.1 *— Two conditions determine whether it is necessary to consider torsional stiffness in the analysis of a given structure:*
(1) the relative magnitude of the torsional and flexural stiffnesses, and
(2) whether torsion is required for equilibrium of the structure (equilibrium torsion) or is due to members twisting to maintain deformation compatibility (compatibility torsion).

In the case of compatibility torsion, the torsional stiffness may be neglected. For cases involving equilibrium torsion, torsional stiffness should be considered.

Lateral Deflections

ACI 8.8.2 *— The selection of appropriate effective stiffness for reinforced concrete frame members* (for lateral deflections) *has dual purposes: to provide realistic estimates of lateral deflection and to determine deflection-imposed actions on the gravity system of the structure. A detailed nonlinear analysis of the structure would adequately capture these two effects. A simple way to estimate an equivalent nonlinear lateral deflection (*δ***em*** *at the top story in IBC 2006) using linear analysis is to reduce the modeled stiffness of the concrete members in the structure.*

The type of lateral load analysis affects the selection of appropriate effective stiffness values.

Deflections from FACTORED lateral loads

For example: Earthquake load in IBC 2006

ACI R8.8.1 *— When analyzing a structure subjected to earthquake events at short recurrence intervals, some yielding without significant damage to the members may be a tolerable performance objective.*

***ACI R8.8.2** — The lateral deflection a structure sustains under factored lateral loads can be substantially different from that calculated using linear analysis in part because of the inelastic response of the members and the decrease in effective stiffness.*

For earthquake loading, a level of nonlinear behavior is tolerable depending on the intended structural performance and earthquake recurrence interval.

The alternative options presented in 8.8.2 use values that approximate stiffness for reinforced concrete building systems loaded to near or beyond the yield level and have been shown to produce reasonable correlation with both experimental and detailed analytical results.

The effective stiffnesses in Option (a) were developed to represent lower-bound values for stability analysis of concrete building systems subjected to gravity and wind loads.

Option (a) is provided so that the model used to calculate slenderness effects may be used to calculate lateral deflections due to factored wind and earthquake loading.

***ACI 8.8.2** — Lateral deflections of reinforced concrete building systems resulting from factored lateral loads shall be computed either by linear analysis with member stiffness defined by (a) or (b), or by a more detailed analysis considering the reduced stiffness of all members under the loading conditions:*

(a) By section properties defined in 10.10.4.1(a) through (c); or

(b) 50 percent of stiffness values based on gross section properties.

***ACI 10.10.4.1** — It shall be permitted to use the following properties for the members in the structure:*

*(a) Modulus of elasticity..................... **Ec** from 8.5.1*

*(b) Moments of inertia, **I***

Compression members:
*Columns **0.70Ig***
*Walls —Uncracked **0.70Ig***
*—Cracked.................................... **0.35Ig***
Flexural members:
*Beams **0.35Ig***
*Flat plates and flat slabs........................... **0.25Ig***

*(c) Area... **1.0Ag***

***ACI R10.10.4.1** — The values of **Ec**, **I**, and **A** have been chosen from the results of frame tests and analyses and include an allowance for the variability of the computed deflections.*

*The modulus of elasticity of the concrete, **Ec**, is based on the specified concrete compressive strength while the sway deflections are a function of the average concrete strength, which is higher.*

The moments of inertia are taken from Reference 10.35,

(MacGregor, J. G., and Hage, S. E., "Stability Analysis and Design Concrete," Proceedings, ASCE, V. 103, No. ST 10, Oct. 1977.),
*which are multiplied by the stiffness reduction factor φ**K** = 0.875.*
*For example, the moment of inertia for columns is **0.875(0.80Ig) = 0.70Ig**.*

These two effects result in an overestimation of the second-order deflections on the order of 20 to 25 percent, corresponding to an implicit stiffness reduction of 0.80 to 0.85 on the stability calculation.

*The moment of inertia of T-beams should be based on the effective flange width defined in 8.12. It is generally sufficiently accurate to take **Ig** of a T-beam as two times the **Ig** for the web, **2(bwh3/12)**.*

*If the factored moments and shears from an analysis based on the moment of inertia of a wall, taken equal to **0.70Ig**, indicate that the wall will crack in flexure, based on the modulus of rupture, the analysis should be repeated with **I = 0.35Ig** in those stories where cracking is predicted using factored loads.*

Deflections from SERVICE lateral loads
For example: Wind load in IBC 2006

***ACI R8.8.2**— For analyses with wind loading, where it is desirable to prevent nonlinear action in the structure, effective stiffness representative of pre-yield behavior may be appropriate.*

***ACI R8.8.1**— For wind loading, it is desirable to maintain elastic behavior in members at service load conditions.*

As with lateral stability analysis of concrete structures (R10.10.4), a factor of 1.4 times the stiffness used for analysis under factored lateral loads is adequate to model effective section properties for lateral deflection analysis under service loads.

So drift can be checked on full inertia properties (modifier=0.7x1.4=1.0 of vertical elements) for service wind loads.

Alternatively, a more accurate level of stiffness based on the expected element performance can be determined.

***ACI R10.10.4.1**— Section 10.10 provides requirements for strength and assumes frame analyses will be carried out using factored loads.*
Analyses of deflections, vibrations, and building periods are needed at various service (unfactored) load levels to determine the serviceability of the structure and to estimate the wind forces in wind tunnel laboratories.
The moments of inertia of the structural members in the service load analyses should be representative of the degree of cracking at the various service load levels investigated.
Unless a more accurate estimate of the degree of cracking at service load level is available, it is satisfactory to use 1.0/0.70 = 1.43 times the moments of inertia given here for service load analyses.

Idea is to use the stiffness properties of cross section based on its expected behavior in particular situation, as mentioned in commentary. Commentary gave the example of analysis under wind and earthquake situations.
In case of wind structure is intended to remain elastic under service loads so use properties based on gross cross section or 1.4 times the reduced cross section properties.

In case of earthquake reduced cross section properties are required at service level to estimate drifts and related p-delta/second effects.

Bottom line: it is complex topic and one have to use assumptions. Even if one is using 0.35 and 0.7 factors to size the cross section of member, structure should still stand provided assumptions are uniform throughout the analysis, as concrete has this ability to distribute moments according to provided reinforcement.

Effect of time period and base shear

Time period increases with reduced stiffness. So time period will increase if we apply low stiffness modifiers in ETABS. For instance applying modifiers like 0.9, 0.8, 0.5, 0.4 and so on will increase the time period as you continue this series.

Effect on long-term deflection?

Reduced stiffness is necessary to capture correctly the effect of long-term deflection.

Axial shortening effects?

See - How to use stiffness modifiers in ETABS in this article.

Other Thoughts

For elastic analysis of frame it is OK to use gross properties based on rectangular section as it is done in ETABS . We provide rectangular beam section properties in ETABS, but cast in place beam has T section in positive region while rectangular section in negative region, so using rectangular section along entire length compensates for that.
Moreover if bottom reinforcement of beam is developed in column, as it is normally done, it increases stiffness of beam in negative region. Amount of reinforcement provided in section also plays its role and we don't know how much reinforcement will be required before starting analysis.

ACI 318-05, Section 10.11 recommendations are associated with slenderness effects and governing flexural deformations, in which case, users should modify EI, which correlates with I33 or I22 for frames and either f11 or f22 for shells.

ACI 318-08, Section 8.8 provides modification factors. No recommendation is made for shear, though users should modify GA when shear walls is expected to experience stiffness degradation upon cracking.

Effect on composite members?

How to use stiffness modifiers in ETABS?

Default settings align shear walls such that their 1-axis is horizontal and their 2-axis is vertical. As a result, the flexural modifier EI should be applied to f22 for wall piers and f11 for spandrels. For columns usually I22 and /or I33, and for beams usually I33 assuming default orientation.

When local axes correspond with default settings, modifiers, and their associated properties, are as follows:

- f12 controls shear behavior through the GA component
- f11 controls flexure through EI
- f22 controls axial behavior through EA

  Typically I initially assign a 1.0 factor to f11, f22, and f12 along with a 0.01 factor to m11, m22, and m12 (so that the shear wall essentially has no out of plane stiffness and acts as a classical in-plane only resisting element). I then run the model and check axial stresses against the modulus of rupture to see if the assumption that the wall is un cracked (f11,f22,f12 = 1.0) was correct. For those portions of the wall that are cracked, I reassign with the lower 0.7 crack factor to f11,f22, and f12.

  http://www.eng-tips.com/viewthread.cfm?qid=141414

  As bending and axial stiffnesses in walls cannot be de-coupled, by reducing f11 and f22, you will reduce the axial cross-section which will lead to problems with axial shortening, and will not represent actual behavior. Therefore, for walls it is best to just modify m11 & m22 to 0.7 for uncracked and 0.35 for cracked wall sections.

  Column shortening in tall structures: prediction and compensation by Mark Fintel, Satyendra Kumar Ghosh, Hal Iyengar

  Relative stiffness is important. It means some elements are less stiff than others and this way forces distribution takes place in the structure.

  Do yourself! Make one span simple beam-column frame in ETABS. Apply some load and exclude self weight. Replicate it and keep default 1.0 modifier on one. Apply a modifier to I3, S