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Seismic design of soft-storey buildings

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Carlo Ruiz Castillo

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Dr. Amado Gustavo Ayala Milián

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JURADO ASIGNADO:

Presidente:	M.I. Hugo Sergio Haaz Mora
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1er. Suplente:	Ing. Marcos Trejo Hernández
2do. Suplente:	M.I. Octavio García Domínguez

Lugar donde se realizó la tesis:

Instituto de Ingeniería, UNAM

TUTOR DE TESIS

Dr. Amado Gustavo Ayala Milián

____

FIRMA

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Abstract

This thesis presents the state-of-the-art on the seismic design and assessment of soft/weak storey buildings, specifically in what regards Building Code's specifications and recommendations for the seismic design of this type of vertical irregularity. A summary of various authors' past research on the topic of buildings with a soft/weak storey irregularity is shown. Afterwards, a short review of the influence of geometric second order effects ($P\Delta$) and cyclic degradation of elements in the global stability and collapse evolution of a framed structure with soft/weak storey characteristics is presented, complemented with collapse-assessment methodologies. The theoretical basis required for the definition of a practical criterion to delimit this type of irregularity is investigated, in order to posteriorly develop a design methodology that properly includes the most relevant aspects on the performance of such buildings. Finally conclusions are drawn as to the present state and future state of the topic.

Resumen

Esta tesis presenta el estado del arte en el diseño y evaluación sísmica de edificios con piso blando/débil, específicamente las especificaciones y recomendaciones de los códigos de construcción a este tipo de irregularidad vertical. Un resumen de estudios previos de diversos autores sobre el tema de edificios con piso/blando débil se muestra. Posteriormente, se muestra una revisión de la influencia de efectos geométricos de segundo orden ($P\Delta$) y la degradación cíclica de elementos en la estabilidad global y la evolución del colapso de una estructura a base de marcos con características de piso blando/débil. Se complementa lo anterior con metodologías para evaluar y delimitar el colapso estructural. Se investigan las bases teóricas requeridas para la definición de un criterio práctico para delimitar este tipo de irregular, para posteriormente poder desarrollar una metodología de diseño que incluya los aspectos más relevantes en el desempeño estructural de este tipo de edificios. Finalmente se extraen conclusiones para el estado actual y futuro del tema.

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1 Introduction

1.1 Background

During the 1985 Michoacán earthquake that struck México City a large number of buildings suffered severe damage or collapsed [Rosenblueth and Meli, 1987], even though it has been proven that these effects were in part due to an exceedance in the earthquake's intensity not contemplated in the Building Code used to design the affected buildings, it also showed limitations on the approaches and criteria at that time, some of which continue to this day [Esteva, 1987]. One of such approaches pertains to a vertical irregularity condition named *soft* or *weak* storey (`sws`) that has the following characteristics:

1. Abrupt lateral stiffness or strength or both interstorey decrease.
2. Excessive mass concentration on some storeys, leading to high ductility demands on a particular storey.

It has been shown that buildings with in-height irregularities perform poorly [Kirac et al., 2011], as their configuration leads to excessive ductility demands on the first storey, this configuration was also responsible for 7700 homes that became inhabitable in California's Loma Prieta earthquake in 1989 and more during the 1994 Northridge earthquake. It also could be responsible for a major part of the 160 000 destroyed or inhabitable homes projected in the San Francisco Bay Area, California due to a strong seism. [Gov.California, 2013]

Traditionally, aesthetics and functionality have dictated higher first storeys , allowing thus for ample car storage or big reception halls, invariably locating the residences or commerce on the upper floors. This configuration is both visually-pleasing and practical, however, it may lead to excessive mass on the higher floors and lower stiffnessess and strengths on the first floor, leading to a lateral discontinuity in the ductility demand when subjected to strong shaking [González et al., 2010]. Said configuration is common even in high seismicity zones, where it has been shown to be prone to collapse, condition which is unacceptable in contemporary seismic engineering.

The `sws` condition is also latent in reinforced concrete (`rc`) frame buildings, whose division walls are structurally joined (by design or as a result of a faulty construction process), adding lateral stiffness and limiting drifts which were not properly accounted for in the building's design.[Díaz, 2008]

Past studies have shown that this peculiar configuration leads to vulnerabilities towards seismic events regardless of the structural system or the materials used [Ruiz and Diederich, 1989].

Building Codes (BC) around the world dictate special special design provisions to protect the building against the undesirable effects of these configurations, either by "penalizing" the design ductility or by setting stricter upper bounds on lateral drifts, the former increasing design base shear, while the latter may lead to the use of infill walls augmenting mass and lateral stiffness. The norms of these codes generally delimit the so-called *structural irregularities* and which modify design parameters to compute the seismic demand, which supposedly guarantees adequate structural performance. Both actions make use of certain hypotheses in order to facilitate the design process of such irregular structues. The validation of these hypotheses a problem of balance, on one hand, it has been proven that elastic analysis methods are often insufficient in order to accurately describe non-linear behaviour of structures under high intensity demands and, on the other hand, the use of rigorous step by step analyses is not yet realistic for practical structural design because they involve concepts not always available to structural engineers besides being costly both in time and computational power.

The aforementioned methods fail to take fully into account the redistribution of internal foces present in the inelastic range of behaviour, even more so for certain irregular building configurations [Ayala, 2001]. Thus, they do not not only ignoring the possible consequences of these effects, but most importantly, they are not directly adress the formation of mechanisms that may lead to the collapse of the structure. Moreover, current force-based design recommendations of rc buildings lack specific capacity design considerations, so strengths required to prohibit non-ductile failures or limited ductile global failure mechanisms such as soft storey sway are not assured [Priestley, 1995].

The present study attempts to compile relevant research on the topic, and addresses multiple BC's specifications and recommendations for the seismic design of sws frames, it is the author's opinion that current approaches to this type configuration lack conceptually coherent design ductilities and do not guarantee adequate structural performance in the event of a high intensity seism.

1.2 Objective

This thesis has as objectives:

1. Explore, through an extensive literature review and evaluation, the possible weaknesses and limitations of current code recommendation for the assessment and design of buildings with a soft/weak storey irregularity.

2. Based on past research results and current concepts of performance-based design, investigate the theoretical basis required for the definition of a practical criterion to delimit the conditions and effects of `sws`.

3. Review the influence of geometric second order effects and cyclic degradation of elements in the global stability evolution and collapse evolution of a framed structure with `sws` characteristics.

It is expected that completion of these objectives will provide useful enough insights to the problem to subsequently develop a design methodology that includes the most relevant research results on the topic.

1.3 Scope

Throughout this thesis, past research results taken from recognized earthquake engineering literature is compiled and their conclusions analysed and compared. Futhermore, the treatment on the topic by the following building codes will be reviewed and evaluated:

- Mexico City Building Code complementary rules for seismic design [GDF, 2004b].
- Eurocode 1998-1 (EN8-1),[ECS, 2004]
- International Building Code,[ICC, 2015]
- National Building Code of Canada.[NRC, 2010]

Structural collapse in `sws` buildings will be studied by reviewing the literature, specifically in what regards influence of geometric second order effects and the cyclic degradation of elements in its global performance. Afterwards, a collapse assessment methodology will be shortly presented. Finally, conclusions will be drawn as to the present state of the art on the topic.

2 State of the art on soft storeys

2.1 What is a soft storey?

Soft storey is the structural condition of a notorious disparity between lateral resistances in a framed structure, while the term *weak* storey refers to lateral stiffnesses. Such conditions are common in the first storey of a building, and are partly due to the 5 guiding principles of modern architecture established in the beginning of the XXth century. Contained in these principles is the so-called *pilotis*, established by Le Corbusier [Jeanneret, 1923], which suggets replacing walls in the first storey by slender columns. This is done in order to give ample space and free transit inside the first storey of the building, in contrast to a paralyzed design (plan paralysé). The former supposedly possesses economical, hygienic and aesthetic advantages, while the latter suffers in these regards. However, seismic response of these type of configurations was not properly considered in the conception of *pilotis*, and major earthquakes from that same century showed that they tend to collapse abruptly. Reconnaissance reports of high intensity seisms (México '85, Northridge '94, Izmit '99) exposed that this condition did not lead to adequate seismic performance as many buildings showed severe damage and others even collapsed [Ruiz and Diederich, 1989, Guevara-Perez, 2012]. Thus, the relation between architectural configuration and adequate structural behaviour in high intensity earthquakes cannot be directly established from geometrical properties or preliminary designs. Topological configuration is a necessary yet not sufficient condition in order to guarantee adequate structural performance.

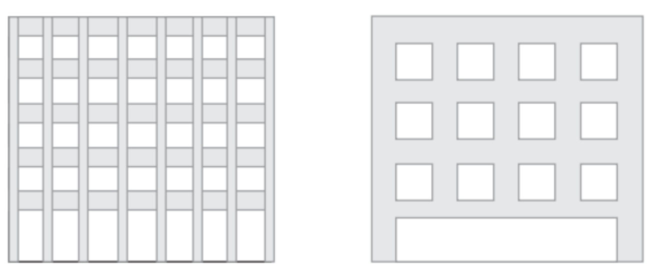

Figure 1: Conceptual drawing of different `sws` configurations

To guarantee said adequate structural behaviour, Building Codes around the world delimit as a first step an *irregularity* condition, in this case the *vertical irregularity.* Very broadly, a building is soft/weak in a storey if there exists:

- abrupt lateral stiffness or strength interstorey decrease,
- excessive ductility demands on a storey during a strong excitations,
- excessive mass concentration on upper storeys.

Evidently, these parameters involve some qualitative definitions, thus, the codes set a somewhat arbitrary threshold to identify the condition. Afterwards, current design methodologies stipulate a "penalization" of these design demands, increasing overall design base shear. However, they fail to address directly the subjacent collapse-mechanism responsible for the inadequate performance, which has been shown to be ruled primarily by geometric second order effects ($P\Delta$), which coupled with cyclic degradation of structural members, governs the dynamic stability condition and the collapse evolution of the system [Ibarra and Krawinkler, 2005].

2.2 Design and assessment of soft/weak storey buildings

2.2.1 Performance-based seismic design

The fundamental objective of past and current seismic design codes is to provide solutions towards the good performance of a building in normal conditions, whilst maintaining its integrity against extraordinary events. Global integrity must be guaranteed in order to preserve human lives, even allowing non-repairable damage to occur. Thus, this design philosophy envisions the structure to undergo large deformations in the event of a high intensity earthquake, sustaining severe amounts of controlled damage, while maintaining an acceptable working condition during low to medium intensity events. Conceptual difficulties arise in establishing a rigorous basis to guarantee *adequate performance* and *global integrity* of a structure, this is currently accomplished in the performance-based design codes by establishing performance objectives.

Performance objectives are thus a selection of design criteria which include: selection of structural system, representative delimitation of seismic hazard, and appropriately chosen indices which are representative of a certain damage state of the structure. The latter criteria are known as "limit states" and set thresholds upon engineering demand parameters such as drift, ductility, etc. Limit states which comply with safety on high intensity seisms are called "ultimate" (`ULS`), while the ones attending performance on normal working conditions are named "serviceability limit states" (`SLS`). Satisfying a certain performance objective is complying with a series of limit states at different levels of seismic demand.

Therefore, the main objective of seismic structural design may be rephrased as follows:

"Provide safety against the occurrence of ultimate limit states, and adequate performance residing below the serviceability limit state."

Table 1: Description of the qualitative definitions of usual Code limit states [Fardis, 2010]

Limit state name	Type	Facility operation	Structural condition	Deformation limit	Seismic action
Operational (OP)	`SLS`	Continued use: non-structural damage may be repaired later	No structural damage	Mean yield value may be reached	Frequent ~70% exceedance probability in service life
Immediate use (IU)	`SLS`	Safe, however normal use is interrupted	Light structural damage, localised bar yield, concrete cracks and spalling	Twice the mean yield value may be exceedeed	Occasional ~40% exceedance probability in service life
Life Safety (LS)	`ULS`	No threat to life during seism, unsafe for normal use, feasible but expensive repair	Significant damage, far from collapse, enough capacity for gravity loads	Safety factor of 1.35 against reaching lower 5%-fractile of plastic rotation capacity	Rare ~10% exceedance probability in service life

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Limit state name	Type	Facility operation	Structural condition	Deformation limit	Seismic action
Near collapse (NC)	`ULS`	Unsafe even for emergency, life safety during earthquake ensured but not guaranteed due to nonstructural element breaking and falling debris	Heavy unrepairable structural damage on the verge of collapse, sufficient strength for gravity loads but not aftershocks	Lower 5%-fractile of plastic rotation capacity may be reached, safety factor of 1	Very rare ~2-5% exceedance probability in service life

2.2.2 Past studies on the soft/weak storey condition

2.2.2.1 Esteva (1987) [Esteva, 1987]

This author studied the nonlinear seismic response of `sws` designed according to the mexican `BC` subjected to the 1985 Michoacán seism. He analysed the influence of the **r** factor (average over-strength factor at the upper stories to that at the corresponding storey) on the ductility demands and overall safety factors. He found that the response of the system is highly sensitive to this design parameter, and it is enhanced strongly by $P\Delta$ effects, leading in some cases to excessive drifts and safety factors. Finally he recommends that more studies should be carried out to cover:

- influence of post-yielding stiffness,
- stiffness and strength degradation of elements,
- frames with infill panels.

2.2.2.2 Ruiz & Diederich (1989) [Ruiz and Diederich, 1989]

These authors studied the seismic performance of `sws` with brittle and ductile infill walls, monitoring the ductility demand at the first storey. They varied the ratios of elastic lateral stiffnesses and strengths and subjected the buildings to the ew component of the ground motion recorded by the accelerograph at the Ministry of Communication and Transportation of the 1985 Michoacán seism. They found that ductility demands are very sensitive to the ratio of the dominant periods of excitation and response, which is closely related to the occurrence of plastic hinges and the yielding or fracture of the infill walls, commenting that large uncertainties tied to the modelling of nonlinear dynamic response systems at their time made it difficult to derive simple rules which delimit safe design strength or stiffness ratios.

2.2.2.3 Bento & Azevedo (2000) [Bento and Azevedo, 2000]

These authors analyzed the influence of the prescribed q factors on the behaviour of soft-storey reinforced concrete structures under seismic excitations with a probabilistic safety checking method based on damage indices. The methodology employed utilized vulnerability functions, which represent the non-linear relationship between the intensity of the actions and the values of the action-effects, quantified in terms of the Miner damage index, ultimately computing the probability of failure. Models considered were planar frames with different characteristics, designed according to Eurocode-8 specifications. They concluded that this kind of structures exhibit less safe behaviour than non-irregular structures, due to the concentration of damage at the soft-storey level and the corresponding excessive drift. They emphasized the importance of the correct consideration of $P\Delta$ effects, and the need for a probabilistic analysis to assess structural safety since the response of this type of structures compared to regular ones differ dramatically with the increase of the seismic intensity, as can be concluded from the vulnerability curves.

2.2.2.4 **Das & Nau (2003)** [Das and Nau, 2003]

These authors conducted an investigation of the definition of vertical structural irregularity based on: stiffness, strength, mass and due to the presence of masonry infill walls. They performed linear and nonlinear transient analyse on an ensemble of 78 special moment-resisting frames (`smrf`) designed with the forces obtained from an equivalent lateral force analysis (`ELFA`) according to the UBC1997's strong column-weak beam criterion and other recommendations therein. Observing that the majority of structures exhibited acceptable performance when subjected to the design earthquake gound motion, however, the ductility demands in the plastic regions increased in the vicinity of the irregularities. They concluded that the restrictions on the applicability of the `ELFA` procedure found in most `BC` s is unnecessarily conservative for certain types of vertical irregularity.

2.2.2.5 **Chintanapakdee & Chopra (2004)** [Chintanapakdee and Chopra, 2004]

These authors studied in-depth the seismic performance of generic strong column-weak beam high ductility special moment resisting frames with three types of vertical irregularities: stiffness, strength and combined stiffness and strength. The influence of this parameters in the response of the structure was studied comparing the median seismic demands computed by nonlinear transient analyses for an ensemble of 20 large-magnitude-small-distance records. They found that introducing a `sws` increases the storey drift demands in the modified and neighbouring storeys and decreases drift demands in the other. Irregularity on the upper storeys has very little influence on floor displacements, in contrast, irregularity in lower storeys has a significant influence on the height-wise distribution of floor displacements.

2.2.2.6 **Rodsin, Nelson, Wilson & Goldsworthy (2006)** [Rodsin et al., 2006]

These authors developed and callibrated fragility curves for a soft-storey column that define their probability of collapse when subjected to excessive drift demands. These curves allow for a more realistic representation of the seismic vulnerability of the building than the conventional approach based on the degradation in the horizontal resistance. They developed an analytical formulation which models shear collapse taking into account the slippage strength along a major diagonal shear crack. Thus, this model predicts the behaviour of a column at the limit of collapse and has been callibrated for columns with low aspect-ratio and that fail in shear or flexure-shear. Furthermore, the model was used to construct fragility curves, useful for assessing the performance of a building in regions of low and moderate seismicity.

2.2.2.7 Fragiadakis (2006) [Fragiadakis et al., 2006]

This author proposes a methodology to evaluate the seismic response of structures with "single storey vertical irregularities" based on Incremental Dynamic Analyses (`IDA`) [Vamvatsikos and Cornell, 2002]. This tool is regarded as state-of-the-art analysis, since it can provide accurate estimates of the complete range of the system's response, including dynamic instability. Limit states can be defined on each `IDA` curve and with some artifice produce the probability of exceedance of said state for a given intensity measure. A more detailed description of the method is done in sec. 3.3.

2.2.2.8 Tena-Colunga (2010) [Tena-Colunga, 2010]

This author studied the definition of the `sws` irregularity condition in the México City Building Code and proposes an alternate definition and delimitation of the condition, based on the substantial reduction of the lateral shear stiffness of one or more resisting frames within a given storey. The analytical models in the study all developed a soft-first-storey response when subjected to the acceleration records that give rise to the design spectra in the Code. He concluded that the simple design recommendations contained in the Code are *somewhat* effective at improving the seismic performance of `sws` irregular buildings *i.e.,* the amplification of the design forces based on a penalization and reduction of the "seismic performance factor". He remarks that the lateral storey stiffness and strength computation method should be explicitly stated in the Codes, for he obtained different seismic behaviours of the same structure when different methods for computing said stiffnesses and strengths were used.

2.2.2.9 Dadi & Agarwal (2015) [Dadi and Agarwal, 2009]

These authors attempted to update the nonlinear modelling of soft storey `rc` frame buildings for performance-based design. They conducted cyclic test on a 1/4 scale prototype soft storey frame building while analytically modelling the failure modes as per ASCE7 and Indian codes' ULS. In this way, they updated the analytical model at three stages of behaviour *i.e.,* linear, nonlinear and failure. They concluded that the nonlinear properties of the reinforcement used in beam components of the frames have a significant influence on the global failure pattern, particularly assuring a flexural failure mode, which is responsible for the ductile response of the system.

2.2.2.10 Summary of past research on sws

- Soft and weak storey buildings exhibit radically different behaviour than regular structures, particularly on the drift demands at the soft/weak storey level, for this reason, performance assessment is of paramount importance in order to assign rational behaviour factors,

- performance assessment of sws buildings has been restricted to few cases, as it is highly dependent on uncertain parameters and are difficult to model correctly, thus, a probabilistic approach should be preferred,

- code designed sws buildings satisfy serviceability limit states reasonably well,

- sws buildings designed with contemporary BC recommendations do not decisively guarantee ultimate limit state compliance nor uniform performance,

- lateral storey stiffness and strength computation procedures should be code-specific and explicit in order to obtain consistent designs within a given framework,

- Mexican code provisions, and other design does prescribing base shear augmenting schemes for sws are *somewhat* effective at improving overall structural performance, although not entirely satisfactory in many cases,

- the contribution of infill walls to the performance of sws buildings has been limited, as their behaviour and influence to overall global stability under intense excitation has not yet been decisively determined,

- geometric second order effects enhance first storey ductility demands greatly and play a major role in their collapse capacity,

- nonlinear properties of `rc` elements significantly influence global instability, thus, adequate modelling is mandatory for predicting their true behaviour under strong excitations,

- the evaluation of the seismic response of a `sws` building up until dynamic instability can be accurately estimated via `IDA`, provided the modelling and the record ensemble is robust.

- limited efforts have been devoted to the development of design methods that incorporate the collapse mechanism of `sws` buildings or the influence of geometric second-order effects directly.

2.2.3 Building Codes on soft/weak storeys

Current building codes (`BC`) prescribing force-based design procedures and recommendations, classify a building to be *vertically irregular* if its initial elastic lateral storey shear strength or stiffness shows a disparity between successive stories, however, the threshold of this characteristic is somehow arbitrary and depends on the calculation procedure used to calculate it which is usually not explicit. Furthermore, stiffness and strength are known to degrade during a high intensity earthquake and as such, are a function of the site itself, this degradation leads to a redistribution internal forces in the inelastic range of behaviour. This degradation drastically alters initial lateral shear strength and stiffness, possibly misleading the computance of design forces and the overall performance of the structure.

In particular, codes include a "penalization" in the design of `sws` due to its irregularity condition *i.e.,* its "seismic performance factor" is reduced, thus incrementing design base shear. The influence of geometric second order effects in the global performance of the structure is treated predominantly on the basis of elastic stability coefficient tresholds, for which they may be completely disregarded if they lie below certain level. If these second order effects are not negligible, they may be taken into account directly by second order analyses, or more frequently, as a modification of first order flexural demands. This approach is not conceptually consistent and further treatment is given in sec.3.1. Cyclic degradation of elements is almost altogether disregarded as their modelling schemes are cumbersome and time consuming.

2.2.3.1 Building Codes and regulations in the United States Due to its political organization this country does not have a single mandatory building code, nor an explicit unified consensus for design and construction of buildings. Codes and Standards are adopted and enforced legally by every State, chosen from so-called *Model Codes* which have no legal status until they are enforced by local government. They seek to provide minimum parameters to ensure public safety, health and welfare. Today the model code that is used predominantly is the International Building Code (IBC).

2.2.3.1.1 International Building Code (2009) The International Building Code [ICC, 2015] is a model code developed by the International Code Council. It was developed to consolidate different existing buildings codes into one uniform code that could be used nationally and internationally to construct buildings. The first editions used as a basis for the seismic design requirements the National Earthquake Hazard Reduction Program (NEHRP) *Recommended Seismic Provisions* [NERHP, 2007], the 2003 version of the NEHRP documents was published as FEMA450 [NEHRP, 2003] , and it was used as the basis for the 2006 IBC edition and the the American Society of Civil Engineers (ASCE) *Minimum Design Loads for Buildings and Other Structures* (ASCE7) [ASCE, 2013], which is adopted by the IBC by reference for most seismic design requirements, be analysed hereafter. It appears that, after all, in the United States there is some consensus on the requirements for seismic design, and it suffices to analyze the ASCE7 standards hereafter in order to get an overview on the sws treatment.

2.2.3.1.2 ASCE7 and NEHRP *Provisions* Implicitly, in these standards the performance-objective for seismic design in the standard has a first basis that structures will have a suitably low likelihood of collapse in rare events (defined as the maximum considered earthquake (MCE) ground motion). A second basis is that life-threatening damage, primarily from failure of nonstructural components in and on structures, will be unlikely in a design earthquake ground motion (defined as 2/3 MCE).

As a first step in understanding how the code treats sws in design, it is necessary to have a general understanding of the analysis and design methodology, for that, some concepts need to be summarized:

1. Risk Category and Importance Factor Known also as *occupancy category*, ranges from I to IV, and expresses the importance of the structure to the community in the event of a high intensity earthquake and is related to the consequences it would bring should a failure occur. Thus, the more the structure qualitatively meets one or more of the following criteria, the higher its importance factor I_e needs to be:

- The building is necessary for the response and recovery efforts immediately following an earthquake,
- The building presents the potential for catastrophic loss in the event of an earthquake, or
- The buildings houses a very large number of occupants or occupants less able to care for themselves than the average.

Generally, residential buildings have a risk category of II, unless they house persons with limited mobility (jails, schools, healthcare facilities) or support lifelines and utilities important to a community welfare amongst others, in which case they receive a category of III. Very rarely will a `sws` be essential to post-earthquake response (e.g. hospital, police station etc.) or house very large quantities of hazardous materials, for which they would receive a category of IV.

2. Seismic design categories (`SDC`) Named $A, B, C \dots F$, they are independent of the structure's properties and represent the seismic risk associated with the intensity of the ground shaking and other earthquake effects the structure will likely experience; with A being the lowest risk and F the highest. These categories allow "to step progressively from simple, easily performed design and construction procedures and minimums to more sophisticated, detailed, and costly requirements as both the level of seismic hazard and the consequence of failure escalate". As such, they act like step functions that trigger on or off requirements, thus, they perform one of the functions of previously used seismic zones. However, they also depend on the building's occupancy and therefore its desired performance. In some sense, they are analogous to the Modified Mercalli intensity scale:

Intensity	qualitative description	SDC
V	no real damage	A
VI	light structural damage	B
VII	hazardous structural damage	C
VIII	hazardous damage to susceptible structures	D
IX	hazardous damage to robust structures	E,F

In the design methodology, `SDC` depends not only on the site but also the soil conditions, thus, in order to determine a structure's `SDC`, it is necessary to determine the so-called S_s, S_1 parameters according to the site, and adjust those values in order to account for the soil conditions that are present at the building's actual construction location. Afterwards, the design earthquake motion is obtained and the design parameters are now labelled S_{DS}, S_{D1} respectively.

(a) Soil site class As has been mentioned, actual site soil conditions are important for determining the seismic design parameters *i.e.,* hard rock and dense soils transmit the seismic waveform with high-frequency (short period) and tend to attenuate shaking with low-frequency (long-period). A comprehensive in-situ geotechnical analysis is allowed in principle, in order to determine the importance of these effects. However, in practice, these effects can be approximated based on the average properties of the soil within 100 feet of the ground surface directly or indirectly via simplified tests such as *Standard Penetration Test* or even a visual and manual classification:

Table 2: Soil classification table [FEMAP-749, 2010]

Site class	Visual and manual classification	Shear wave velocity	Blows SPT	shear strength
-	General Description	ft/s	number	psf
A	Hard rock	>5000	-	-
B	Rock	2500-5000	-	-
C	Very dense soil and soft rock	1200-2500	>50	>2000
D	Stiff soil	600-1200	15-50	1000-2000
E	Soft clay soil	<600	<15	<1000
F	Unstable soils	-	-	-

Typically, A is only found in the eastern US, while the western states include various volcanic deposits, sandstones, shales and granites that are commonly B or C. The `NEHRP` *Recommended Seismic Provisions* permit any site to be categorized as D unless there is reason to believe that it would be more properly classified as E or F.

(b) Design Ground Motion Is defined by an acceleration response spectrum characterized by the following parameters:

- S_{DS} short-period design response acceleration (in units of percent g)
- S_{D1} one-second period design response acceleration, (in units of percent g)

- T_s transition period from constant response acceleration to constant response velocity, in units of seconds,
- T_L transition period from constant response velocity to constant response displacement, in units of seconds.

Then, Hazard Maps provide S_s, S_1 mapped values of `MCE` spectral acceleration for reference soil conditions, afterwards, the design parameters are computed as follows:

$$S_{DS} = \frac{2}{3} F_a S_s \tag{1}$$

$$S_{D1} = \frac{2}{3} F_v S_1 \tag{2}$$

The F_a, F_v coefficients are related to the Site Class, and indicate the relative amplification or attenuation effects of site soils, and are presented in form of tables.

3. Selection of structural system and system parameters. The assumptions for which the design procedures were developed, which include uniform configuration of structures, may become invalid if said regularity is not met within the selection of the structural system. These conditions are termed "irregularities" and may trigger requirement of more exact methods of analysis that better reflect the real behaviour of the irregular distribution of forces and deformations in the structure during strong earthquake shaking. Some types of irregularities are prohibited for `SDC` E or F.

   Two basic types of irregularity are found in the code:

   - Vertical irregularity,
   - Horizontal irregularity.

   This thesis will only be concerned with the former, which include the following:

   Stiffness soft-storey irregularity this occurs when the stiffness of one storey is substantially less than that of the stories above. This commonly occurs at the first storey of multistorey frame buildings when the architectural design calls for a tall lobby area. It also can occur in multistorey bearing wall buildings when the first storey walls have a number of large openings relative to the stories above. It exists if stiffness of any storey is less than 70% of the stiffness of the storey above or less than 80% of the average stiffness of the three stories above and is termed **1a**.

Extreme stiffness soft-storey irregularity As its name implies, this is an extreme version of the first soft-storey irregularity. This irregularity is prohibited in Seismic Design Categories D, E and F structures. It exists if stiffness of any storey is less than 60% of the stiffness of the storey above or less than 70% of the average stiffness of the three stories above and is termed **1b**.

Weak-storey irregularity occurs when the strength of the walls or frames that provide lateral resistance in one storey is substantially less than that of the walls or frames in the adjacent stories. This irregularity often accompanies a soft-storey irregularity but does not always do so. It exists if the lateral strength of any storey is less than 80% of the strength of the storey above and is termed **5a**.

Extreme weak-storey irregularity as its name implies, this is a special case of the weak-storey irregularity. Structures with this irregularity are prohibited in Seismic Design Categories D, E and F. It exists if the lateral strength of any storey is less than 65% of the strength of the storey above and is termed **5b**. These shall not be over 9m in height. This limit does not apply where the "weak" storey is capable of resisting a seismic force of Ω_o times the design force.

There is an exception to these rules: the irregularity condition does not exist if no storey drift ratio is greater than 1.3 times the drift ratio of the storey above.

System parameters $(R, \Omega_o, C_d, \rho I_e)$: response modification coefficient, system overstrength parameters, deflection amplification failure, redundancy factor and seismic importance factor respectively, are used in order to modify the seismic design forces and computed drifts in order to estimate more accurately the system's response to a high intensity earthquake, taking into account structural irregularities and the inelastic response of the system based on said configuration:

R accounts for the seismic force **reduction** due to inelastic behaviour of elements, the purpose of this factor is to provide a rational re-

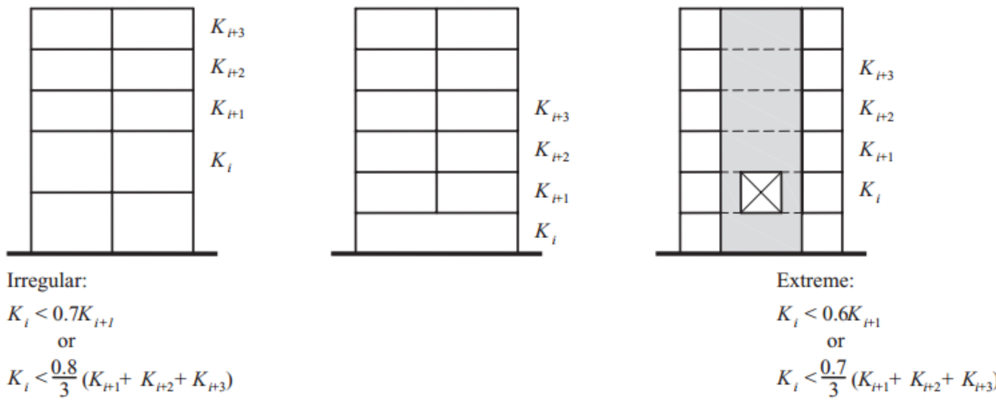

Figure 2: Conditions for soft or extreme soft-storey according to `ASCE7`

lationship between the elastic response spectrum demand and the inelastic response reduction capabilities of the structural system.

Ω_o is the relation between the final yield to the first significant yield and increases the required seismic forces

C_d is used to compute final inelastic displacements and is analogous to the real ductility of the structure.

ρ as its name implies, it tries to encourage the design of more redundant structures, with a greater number of elements that provide lateral force resistance. For practical purposes and for structures assigned `SDC` of B or C, a value of unity is permitted.

I_e importance factor defined at the beginning of the design process, equal to 1.00 if the risk category is I or II, equal to 1.25 if the risk category is III and 1.50 if the risk category is IV. This factor **amplifies** seismic design forces.

Pictorially:

As this work treats `SMRF`, a summary of the system parameters found in Table 12.2-1 is presented:

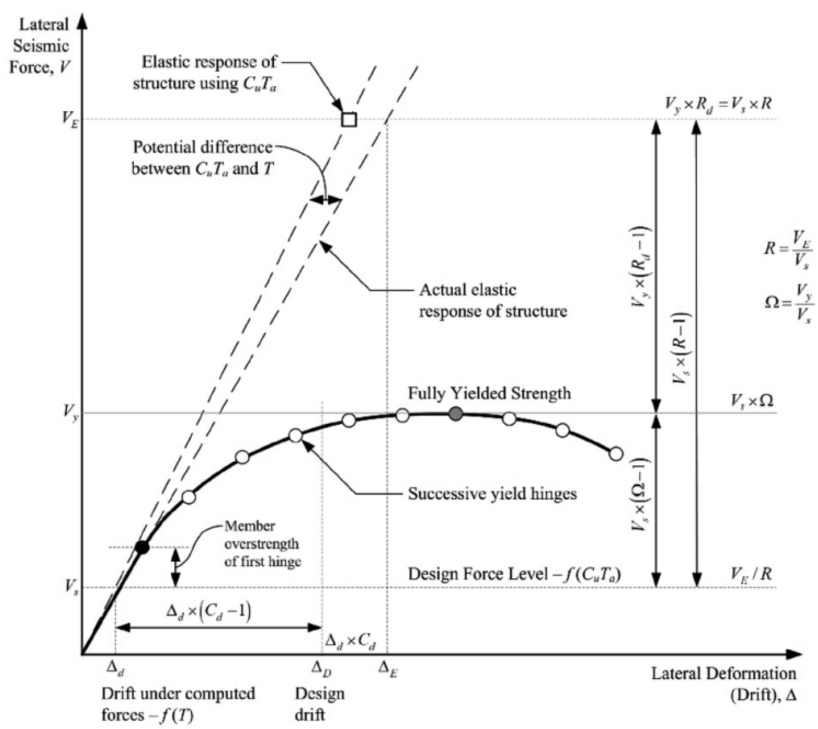

Figure 3: System parameter definitions from a pushover curve.

Type of MRF	R	Ω_o	C_d	Structural system limitations	by category	and	building height limit	
				B	C	D	E	F
Special RC moment frames	8	3	5.5	NL	NL	NP	NP	NP
Ordinary RC moment frames	3	3	2.5	NL	NP	NP	NP	NP
Intermediate RC moment frames	5	3	4.5	NL	NL	NP	NP	NP

Where NL, NP mean not limited and not permitted respectively.

4. Analysis procedures for sws On the basis that it is not possible to have an extreme sws with SDC of D, E or F, and it is very unlikely that the risk occupancy is III or IV, we are left with regular stiffness irregularity type **1a** for SDC of D, according to Table 12.6-1, Equivalent Lateral Force Analysis (ELFA) procedures are allowed as long as the building is less than 48 m high (16 storeys) or . Dynamic analyses such as RSA or step-by-step methods are always allowed, insofar as it is proven that the mathematical modelling was properly constructed according to 12.7.3. As a general rule, design forces shall be multiplied by the factor I_e/R and the computed drifts by C_d/I_e

   Basic load combinations are for allowable stress design:

$$(1.2 + 0.2S_{DS})D + \Omega_o Q_E + L + 0.2S$$
$$(0.9 - 0.2S_{DS})D + \Omega_o Q_E$$

   Where Q_E stands for the earthquake effect on the structure computed with ELFA or RSA

   The ELFA differs from conventional Code procedures at it includes an exponent based on the period of the structure in order to account for the variation of the modal shape:

$$F_i = \frac{w_i h_i^k}{\sum_i w_i h_i^k} \frac{S_{DS}}{\frac{R}{I_e}} \sum w_i \tag{3}$$

Where $k = 1$ if $T \leq 0.5$, equal to 2.0 if $T \geq 2.5$ and $0.5T + 0.75$ if it lies in between those values.

(a) Allowed drifts As mentioned before, computance of the interstorey drift taking into account the structure's ductility is done via the following expression:

$$\delta = C_d \frac{\delta_e}{h_{sx} I_e} \tag{4}$$

Where δ_e is the elastic displacement in the horizontal direction, h_{sx} the storey's height below level x. Table 12.12-1 shows that for structures with risk category of I or II, the maximum allowed drift is equal to 0.020 h_{sx}/rho .

(b) P-Delta effects and structural stability According to 12.8.7, second-order effects may be neglected if the "stability coefficient" θ at each storey is less than or equal to 0.10:

$$\theta = \frac{P_x \delta}{V_x h_x C_d} \tag{5}$$

Where P_x is the total vertical design load at and above level x.
The stability coefficient shall not exceed:

$$\theta_{\max} = \frac{0.5}{\beta C_d} \leq 0.25 \tag{6}$$

Where β is the ratio of shear demand to capacity for the storey between levels x and $x - 1$.
If the coefficient is between 0.1 and $\theta_{\max}$, displacements and forces shall be divided by $1-\theta$. That means a design that includes larger forces and larger drifts.

(c) Modal Response Spectrum Analysis In light of the inapplicability of the `ELFA` to high `sws` structures, it is necessary to illustrate the process of analysis via a dynamical method, for the sake of brevity, the following basic steps must be taken:

- Compute modal properties for each mode (frequency, shape, modal participation factor, effective modal mass)
- Determine the number of modes to be used in the analysis (enough to capture 90% of the mass in each direction)
- Using a compatible ground motion spectrum, compute the spectral accelerations for each mode.
- Multiply spectral accelerations by modal participation factor and I_e/R
- Compute modal displacements for each mode
- Compute elements forces for each mode
- Combine statistically the modal displacements via SRSS or CQQ to determine total system displacements
- Combine statistically the component forces to determine final design forces
- Multiply drifts by C_d/I_e and $1/(1-\theta)$ if necessary.
- Detail members for strength and ductility requirements

2.2.3.2 Canadian Building Code (NBCC) The contents of this Code follow a similar trend to that found in the general design methodology in `ASCE7`. As such, only a brief sketch will be given, the exact parameters are given in detail in 2.2.4.

Section 4.1.1.3.1 establishes that buildings and their structural members, shall be designed to comply with the limit states which it prescribes and divides buildings in 4 categories by importance, assigning an I_E (importance factor due to earthquake) corresponding to ultimate limit state design:

low 0.8

normal 1

high 1.3

post-disaster 1.5

Section 4.13.1 distinguishes three types of limit states:

Ultimate limit state which concern safety and is defined as an exceedance in the load carrying capacity of a structure or component, either by overturning, sliding, fracture, amongst others.

Serviceability limit state which may restrict the use and occupancy of the building. Factors involved are: deflection, vibration, permanent deformation and local structural damage such as cracking.

Failure limit state concerns fatigue and failure due to repeated loading.

2.2.3.2.1 Seismic force resisting system Seismic force resisting system (`sfrs`), introduced in section 4.1.8.3 as part of the structural system that has been considered in the design to provide withstand full earthquake forces and effects. It is stated that all structural framing elements not considered to be part of the sfrs must have sufficient non-linear capacity to support gravity loads with calculated deflections, furthermore, **stiff elements shall be separated from the `sfrs`** and shall not be used to resist earthquake forces. It is evident that they should be taken into account in computing the period of building. Furthermore, section 4.1.3.3. states that sway effects by vertical loads acting on the structure in its displaced configuration shall be taken into account in the design of buildings and structural members (design for $P\Delta$ induced moments.)

2.2.3.2.2 Vertical irregularities The definition corresponds closely to the `IBC` definition:

***Vertical stiffness irregularity* or soft-storey** is considered to exist when the lateral stiffness of the `sfrs` in a storey is less than 70% of the stiffness of any adjacent storey, or less than 80% of the average stiffness of the three storeys above or below.

***Discontinuity in capacity* or weak-storey** is considered to exist when the storey shear strength in a storey is less than that in the storey above. The storey shear strength is the total strength of all seismic-resisting elements of the `sfrs` sharing the storey shear for the direction under consideration. Buildings with this type of irregularity are not permitted except for very low seismic zones, even then, the design forces must be 1.5 times the elastic response with no ductility or overstrength force reductions.

2.2.3.2.3 `ELFA` and `RSA` Spectral accelerations coefficients are obtained form uniform hazard spectra taken from seismic hazard maps with a 2% probability of exceedance in 50 years (2500 year return period). They are a function of the site class (analogous to `ASCE7`).

`ELFA` is limited to structures with irregularity such as `sws` with a total height of less than 20 m. Seisms shall be considered as acting on the `sfrs` bidirectionally with factors of 1 and 0.3 for each case. Base shear is computed by:

$$V = \frac{S_a(T) M_v I_e W}{R_d R_o}$$

Wherein R_d, R_o are the ductility and over-strength factors respectively, and M_v is factor that takes into account higher mode effects on base shear (taken from Table 4.1.8.11). Forces at storey x by the equivalent method are computed as follows:

$$f_{xi} = \frac{(V - 0.007 T_a V) w_i h_i}{\sum_i w_i h_i}$$

Where T_a is the fundamental period of the structure.

2.2.3.2.4 Allowed drifts The largest interstorey drift allowed at any level are $0.025h_s$, accordingly, the computed values from the elastic analysis shall be multiplied by $R_d R_o / I_E$.

2.2.3.3 Eurocode (2004) Eurocodes are the standards for structural analysis and design inside the European Union. This thesis will be concerned with volume 8, first section:

"Eurocode 8: design of structures for Earthquake resistance.", *EN 1998-1: General rules, seismic actions and rules for buildings*

Its main objectives are:

1. Protect human life.
2. limit damage,
3. provide fully operational structures, important for civil protection.

2.2.3.3.1 Overview of the design methodology Its fundamental requirements are **no-collapse** (local or global), and the retention of structural integrity and residual load bearing capacity after a high intensity event. For ordinary structures this requirement should be met for a reference seismic action with a 10% probability of exceedance in 50 years *i.e.,* 475 years return period. The damage limitation requirement specifies that the structure should withstand a more frequent seismic action without damage and avoid limitations of use with high costs. For ordinary structures a 95 years return period is specified.

Eurocode 8 follows a contemporary performance-based design methodology based on design forces. It classifies the structure into importance classes, assigning a higher or lower return period based on the classification.

The standard procedure in EC8 is force-based design subjected to the results of linear elastic analysis (static with lateral forces, or modal response spectrum), for the elastic spectrum reduced by the behaviour factor q . In buildings designed for energy dissipation (*ie.* those of ductility classes M and H), design also aims at controlled inelastic response, by preventing storey-sway mechanisms and brittle failure modes through capacity design of members, and by detailing regions intended for energy dissipation (plastic hinge or "critical" regions) for ductility and deformation capacity.

2.2.3.3.2 Limit states Limit states require checking the following parameters:

- Resistance
- Ductility
- Equilibrium and stability
- Foundation stability
- Seismic joints

For the sake of brevity and the scope of the work only the first three will be covered.

1. Ultimate limit state In order to accept the design as satisfactory, compliance of the following is necessary:

   (a) Reliability differentiation Analogous to importance classes, in operational terms the reference seismic action is multiplied by a δ_1 value (importance factor) assigned from table 4.3 that ranges from 0.8 to 1.5

   Both structural elements and the structure as a whole possess adequate ductility.

   It is also stated that: "In multi-storey buildings formation of a soft-storey plastic mechanism shall be prevented, as such mechanism might entail excessive local ductility demands in the columns of said storey." In order to satisfy this requirement the following condition should be satisfied at all joints of the primary and secondary seismic beams with primary seismic columns:

$$\sum M_{\mathrm{R\ col}} \geq 1.3 \sum M_{\mathrm{R\ beam}}$$

Compute the moments more rigorously, they shall be obtained at the centre of the joint.This expression is analogous to the Mexican Code `SMRF` requirement.

(b) PΔ effects and stability Requires that the structure shall be stable, including overturning or sliding, in the seismic design situation. Although it is not specified how to verify this condition or compute the global stability of the system.

$P\Delta$ effects shall be ignored if:

$$\frac{\delta \sum P}{H \sum V} \leq 0.10$$

Where:

- $\sum P$ is the total gravity load at and above the storey considered in the seismic design. Else they shall be approximated multiplying the relevant seismic action effects by a factor equal to $1/(1-\theta)$.

2. Damage limit state This limit state is considered to have been satisfied, if, under a seismic action having a larger probability of occurrence that the design seismic action corresponding to the "no-collapse requirement", the interstorey drifts are within the following bounds:

(a) For buildings having non-structural elements of brittle materials attached to the structure:

$$\delta\nu \leq 0.005H$$

(a) For buildings having ductile non-structural elements:

$$\delta\nu \leq 0.0075H$$

Where ν is a reduction factor that takes into account the lower return period of the seismic action associated with the damage limitation requirement. (Recommended: $\nu \in [0.4, 0.5]$)

2.2.3.3.3 Design principles

- Structural simplicity;
- Uniformity, symmetry and redundancy;
- bi-directional resistance and stiffness;
- torsional resistance and stiffness;
- diaphragmatic behaviour at storey level;
- adequate foundation.

Further elaboration on the second point states: "Uniformity in the development of the structure along its height is also important, since it tends to eliminate the occurrence of sensitive zones where concentrations of stress or large ductility demands might prematurely cause collapse." and also "A close relationship between the distribution of masses and the distribution of resistance and stiffness eliminates large eccentricities between mass and stiffness".

2.2.3.3.4 Criteria for structural regularity Regarding the criterion for regularity in elevation: this criterion is satisfied if "both the lateral stiffness and the mass of the individual storeys remain constant or reduce gradually, without abrupt changes, from the base to the top".

If this criterion is not satisfied, the behaviour factor that can be used for design purposes must be reduced by a coefficient k_r , reflecting the regularity in elevation, which is equal to 0.8 in the case of non-regular structures. As the k_r coefficient is equal to 1.0 in the case of regular structures, this means that, in the case of irregular structures, the seismic coefficient or the response spectrum ordinates are affected by a factor equal to 1.25 (1/0.8) regardless of the type and severity of the irregularity. For instance, the behaviour factor is the same regardless of corresponding to a soft storey or a setback structure. This shows that there is a need for a clearer characterisation of structural irregularity and that different design procedures or design parameters should be used according to the type or severity of the irregularity and that some types of irregularities should even not be allowed.

Building structures are categorized into being regular or non-regular for the purpose of seismic design. Vertical irregularity significantly affects a behaviour factor q.

In particular, the consequences of structural regularity on seismic analysis and design found on Table 4.1 are:

Regularity		Allowed simplification		Behaviour factor
Plan	Elevation	Model	LE-Analyses	(for the analyses)
x	x	Planar	Lateral force	Reference value
x		Planar	Modal	Decreased value
	x	Spatial	Lateral force	Reference value
		Spatial	Modal	Decreased value

The use of lateral force analyses in vertically irregular frames is **forbidden**, and in particular, for irregular buildings in elevation, the decreased values of the behaviour factor are given by multiplying the reference values by 0.8.

1. Effective stiffness and infill walls In concrete buildings, the stiffness of the columns should be evaluated taking into account the effect of cracking. Such stiffness should correspond to the initiation of yielding of the reinforcement and *may be taken as half of the corresponding stiffness of the uncracked elements*.

   The code considers brick masonry infilled `RC` frames as dual systems, with three ductility classifications.

   Those which contribute significantly to the lateral stiffness and resistance of the building should be taken into account (procedure included in 1.4 Provisions for walls).

   The use of concrete walls is promoted in EC8, through:

   - the low drift limits, which are difficult to satisfy with concrete frames alone – especially as the

   cracked stiffness of concrete members is used,

   - the high q -factors provided for dual and wall systems, and
   - the exemption of columns from capacity design in flexure at beam-column joints, when walls resist at least 50% of the lateral force.

   To fully exploit their potential in preventing soft storey mechanisms, ductile concrete walls should be designed and detailed not only for a large plastic hinge rotation at the base, but also against premature flexural yielding or shear failure elsewhere along their height.

2.2.3.4 México City Building Code The Code [GDF, 2004a], revised and published in 2004 is comprised by a Main Body adressing general design recommendations [GDF, 2004d], and complemented by an Annex [GDF, 2004c] which contains more analytical depth and specifically addresses structures built upon lakebed soils. The Code's performance objective includes the following limit states (Main Body):

Ultimate limit state (collapse-prevention) is any combination of actions that compromise local or global stability and deny the capacity to carry loads. The upper bound on the storey drift depends on whether infill walls and other non-structural elements are joined to the main seismic force resisting system.

Restricted movement $\delta \leq 0.006$

Free movement $\delta \leq 0.012$

Where δ is the interstorey drift.

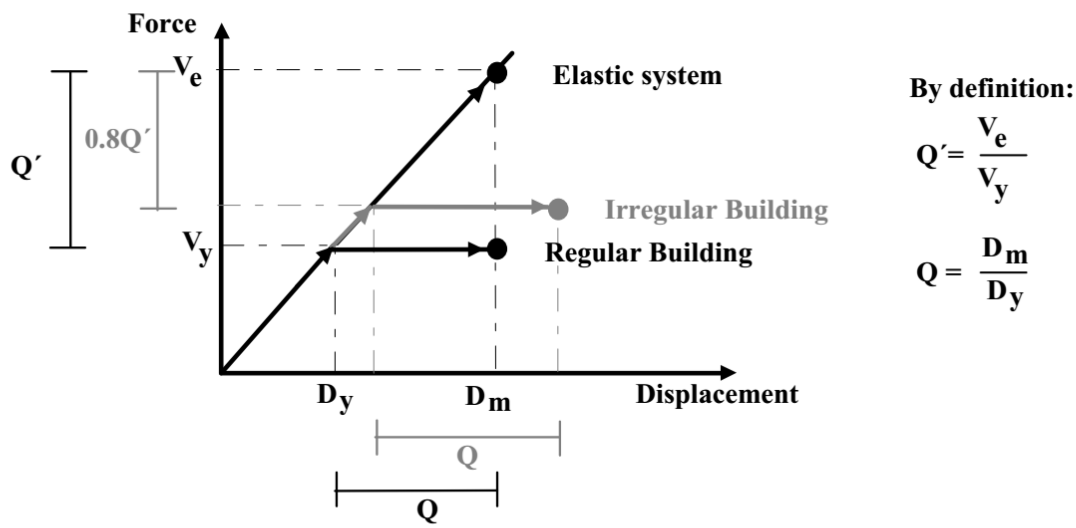

Figure 4: Implicit seismic behaviour factor and ductility definitions in the Mexican code [Tena-Colunga, 2010].

2.2.3.4.1 Seismic behaviour factor and ductility

The "seismic behaviour factor" Q ranging from 1 to 4 included in the main body (fig. 4), by which total design forces are reduced, this is analogous to global ductility. In order for a structure to be granted $Q = 4$, it must comply with the rules set by section 5.1. "Seismic behaviour factor" [GDF, 2004b], these rules dictate that every element must be designed with a ductile failure mechanism and that frames should be the sole contributor to seismic strength, non-structural elements should not joined to the main structural system. Also, shear storey strength must be uniformly distributed and must not differ more than 35 percent in any storey, this strictly rules out a soft storey. Therefore the maximum Q allowed for a `sws` is 3.

2.2.3.4.2 Treatment of vertical irregularities

The soft/weak storey condition is explicitly recognized as an unwanted structural configuration, which is penalized in design by increasing seismic demands as a function of its stiffness or strength disparity. There is also no explicit mention of the method for computing lateral resistances or stiffnesses.

Soft storey condition is addressed in section 6.1 "Regularity conditions - Regular structures" tenth precept [GDF, 2004b]:

"Neither storey stiffness nor shear-strength differ in more than fifty percent from the storey below, the last storey is exempt from this consideration."

Furthermore, Section 6.2 "Irregular structures" states that a building which does not comply with two or more of the above precepts is considered *strongly irregular*. Section 6.3.2 "Strongly irregular structures"explicitly aimed at soft storeys defines strong irregularity as:

"Storey strength or stiffness exceeds in more than 100 percent of the storey below."

Section 6.4 "Irregularity Correction" indicates the modification to the Q' parameter due to these irregularities:

Soft storey only	$Q' = 0.9Q'$
Soft storey plus one or more not met conditions	$Q' = 0.8Q'$
Extreme soft storey (strongly irregular)	$Q' = 0.7Q'$

2.2.3.4.3 Allowed analyses

1. Equivalent static analyses These can be employed to all kinds of structures if they meet the following criterion:

    - Height ≤ 30 m, if irregular ≤ 20 m.

These bounds can be extended to 40 and 30 meters respectively if the structure is located in bedrock. (Zone I).

The static lateral force acting on the centre of mass, corresponding to the i -eth storey is computed by:

$$F_i = \frac{w_i h_i c \sum_i w_i}{Q'(T) \sum_i w_i h_i}$$

Wherein c corresponds to the plateau's acceleration spectral ordinate, $Q'(T_1)$ to the force reduction factor which is a function of the fundamental vibration period of the structure, T, h, w to the storey's height and weight respectively.

2. Modal analyses If the simplified procedure is not applicable or a more rigorous analyses is desired, design spectra are provided which are a function of the microzonation and the "seismic behaviour factor" only. Modal responses shall be combined with the `SSQ` rule and corrected if the base shear does not exceed a minimum value.

3. Other analyses The Code allows the use of more refined dynamic step-by-step analyses methods with real or simulated records, as long as four or more independent and intensity compatible records are used, and the non-linear behaviour of the structure and its uncertainties are taken into account somehow.

4. Treatment of P-Delta effects As customary, they shall be ignored if $\delta \leq V/W$ on any storey, where V, W are the storey's shear and weight above it.

2.2.3.4.4 Annex A This Annex [GDF, 2004c] attempts to make the whole performance-based design process more transparent in terms of a contemporary force-based methodology. It provides clearer and more specific seismic demand computance and shows the dual limit-state design philosophy based on performance criteria for Total Operational and Life Safety performance-objectives, it also includes a more detailed micro-zonation that allows for a more specific and detailed determination of said seismic demands. It also includes the overstrength factor R absent in the main body, this factor is a function of the fundamental vibration period of the structure. The alternate and new design procedure included contains solid conceptual and empirical bases for computing the design spectra taking into account soil-structure interaction. The new elastic spectra pretend to represent in a more realistic manner the seismic design levels, and the consideration for the spectral ordinate reduction for concepts such as ductility and over-strength. Alternate design spectra feature soil-structure interaction and more refined computation of the spectral ordinate c and site specific spectral periods T_b, T_a based on the soil strata configuration and microzonation. This ordinate shall also be reduced by $Q'R$ and the displacement multiplied by Q.

The performance-objective for a reinforced concrete `SMRF` with $Q = 3, 4$ establishes the following limit states:

SLS $\delta Q'R/7 \leq 0.004$

ULS $\delta QR \leq 0.03$

2.2.4 Summary table of contemporary `BC` design methodologies

A summary in the form of a table is presented, wherein the maxima or minima of the allowed design values for a framed structure with a soft-weak storey vertical irregularity is shown, and the allowed procedures and considerations are briefly sketched. The maximum allowed values for site classes or importance factors (that increase design forces) are shown, finally, the theoretically most unfavourable case of the combined values is shown, in order to get a general grasp and make a quick comparison on the design specifications.

Table 4: Table summarizing `BC` specifications towards the design of `sws`

BC	weak-storey extreme weak-storey	soft-storey extreme soft	importance or risk category	site class or equivalent	seismic reduction factor for `sws`	ductility or equivalent	overstrength or equivalent	maximum design force maximum design force
RCDF	50%	50%	≈ 10% increase	microzonation	Q' = 0.9Q `sws`	Q ∈ [1,4]	R = 2	$\frac{1}{0.8(2)} = 0.714$
	100%	100%	gravitational forces for type 1a	I,II,III	Q'= 0.8Q `eswfs`			
EN-8	NC	NC	$\delta_1 \in [0.8, 1]$	A,B..F	$k_r = 0.8q$, $q_{min} = 1.5$	q ∈ [1.5,3]	$\alpha_R = 1.5$	0.8333
	NC	NC	for type I, II		regardless the severity	high (H) = 3	I	
	I, scwb	I, scwb			of the irregularity	medium (M) = 1.5		
IBC	70% or 80% mean of 3 storeys	70 % 80 or % 3 mean of storeys	I,II	A,B,C	R = 8	$C_d = 5.5$	$\Omega_o = 3$	0.043 ?
	60% or 70% mean of 3 storeys	60 % 70 or % 3 mean of storeys	$I_e = 1.0$					

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	extreme weak-storey	extreme soft	risk category	equivalent	factor for sws	or equivalent	or equivalent	maximum design force
NBCC	70% or 80% mean of 3 storeys	70% or 80% mean of 3 storeys	I,II,III,	A,B..F	$R_d = 4$	$R_d = 4$	$R_o = 1.7$	0.0191
	60 % 70 or % 3 mean of storeys	60 % 70 or % 3 mean of storeys	$I_E = 1.3$					

Table 5: Continuation of table summarizing BC specifications towards the design of sws

BC	infill-walls separation and computation	extra conditions or limitations for smrf s	ELFA or height limitation	PΔ with consideration	soil-structure interaction	lateral k for elastic analyses	drift ULS $\leq$	drift SLS $\leq$
RCDF	NR, NC	$\sum M_c \geq 1.5 \sum M_b$ scwb detailing plastic hinge regions	20 m	$\delta_{\mathrm{i}} \leq 0.08 \mathrm{QV_i/W_i}$ NC	NR, E , I	NC 50% brute stiffness due to cracking	0.03/QR	0.028/Q'R
EN-8	Required, E	$\sum M_c \geq 1.3 \sum M_b$ scwb detailing plastic hinge regions	forbidden	$\delta \leq 0.10\ \mathrm{V_i/W_i}$ multiply seismic actions by $1/(1\text{-}\theta)$	NR, I	NC 50% brute stiffness due to cracking	$\delta\nu \leq 0.0075$H brittle non-structural $\nu \in$[0.4,0.5]	0.005
IBC	NR, NC	specify redundancy factor	48 m	$\theta = 0.1$	NR, I	NC,	$0.02\mathrm{h_{sx}}$	NC

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	separation and computation	or limitations for `smrf` s	or height limitation	with consideration	interaction	for elastic analyses	$\leq$	$\leq$
		normally $\rho = 1$ when using Ω_o in analyses	includes higher modes implicitly	multiply seismic actions by $1/(1\text{-}\theta)$				
NBCC	Required, I	$\sum M_c \geq 1.5 \sum M_b$	20 m includes higher modes implicitly	$\theta = 0.1$ multiply seismic actions by by $1/(1\text{-}\theta)$	NR, I	NC	$0.025h_s$	NC

I indirectly

E explicit

NA not allowed

NR nor required

NS not shown

NC not clear

ELFA equivalent lateral force analysis

scwb strong column-weak beam criterion

2.2.5 Effects of BC provisions on collapse capacity

There is consensus regarding the avoidance of structural collapse by the compliance of a predefined ULS, even at the cost of non-repairable damage. Structural collapse or *global instability* is defined qualitatively as the failure of a system's load carrying capacity, experience has shown that certain classes of flexible regular framed structures exhibit a tendency towards the so-called *sidesway collapse* [López et al., 2014]. Sidesway collapse –also known as incremental collapse– is a consequence of the storey's lateral strength or stiffness reduction induced by large displacements, such process is accelerated by the cyclic deterioration of its components [Ibarra and Krawinkler, 2005]. The strong column-weak beam (scwb) criterion objective is, among others, to avoid storey mechanisms that lead to this type of collapse, and to achieve better distributed failure ones. Studies have shown that the influence of the design base shear is of second importance to the collapse capacity of a ductile building compared to this parameter. [Haselton and Deierlein, 2007]. However, a comprehensive study of those recommendations on specific vertical irregularities has not been fully carried out. Furthermore, P-Delta effects predicted by current approaches are all less that the statistics mean values, which indicate that the treatment of these effects is underestimated according to current seismic codes, moreover, the influence of these effects in the inelastic range differ much from the elastic case, revealing that simply extending elastic approaches to inelastic range is inappropriate. Therefore the stability index should be selected according to the structure's response state. [Wei et al., 2012].

As a remark, it is not possible to identify dynamic instability (that is a necessary condition for sidesway collapse) via elastic analyses, since it also depend on the frequency content of the signal (inertial and dampening forces may stabilize the system). [Bernal, 1998] Because of this, it is of utility to study this phenomenon and the effects that may lead to it more closely.

3 Structural collapse in soft/weak storey buildings

In a general sense, collapse refers to a system's failure in its capacity to carry vertical loads. Sidesway or incremental collapse (fig. 5)comes as a result of dynamic instability, which can be defined as [Bernal, 1998] :

Disproportionate system response to a dynamic loading's small variation of intensity in a lapse of time.

This occurs when a hypostatic conditions arises in a transient analyses, that is, when its stiffness matrix is not positive definite [López, 2015], and can be identified numerically in transient analyses provided the structure is appropriately modelled. [López et al., 2014]. However, dynamic instability also depends on the properties of the seismic demand, thus, numerical instability is a necessary but not sufficient condition for collapse, requiring the use of nonlinear dynamic analyses for its identification. Furthermore, instability depends highly upon the hysteretic rules that govern local element behaviour, and for certain classes of framed buildings, is governed by $P\Delta$ effects [Adam and Jäger, 2011a].

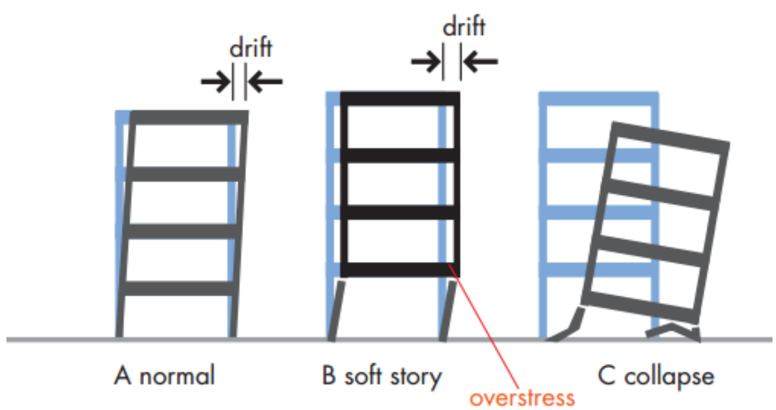

Figure 5: Sidesway-collapse of a soft-first-storey building

3.1 Influence of P-Delta effects

The influence of these effects on the collapse capacity of buildings have been studied by analysing the behaviour of `sdof` systems under various assumptions and methods, such as energy approaches investigated by MacRae and Kawashima [MacRae and Kawashima, 1995], the influence of the negative post-yield stiffness and the dynamic behaviour of these systems was studied by Miranda and Sinan [Miranda and Sinan, 2003], presenting an empirical minimum lateral strength requirement equation. Several authors' have tended to study $P\Delta$ and collapse separately, however, it has been pointed out that they are closely related [Adam and Jäger, 2011b]. Therefore, as a first step in understanding the influence of these effects on the collapse capacity, let us consider a `sdof` system which suffers from gravitational effects at every instant (fig. 6). The linearized form of the nonlinear equation of motion, as a function of the angle ψ of the oscillator's mass with respect to a vertical axis, can be expressed as follows [Adam and Jäger, 2011b]:

$$mh^2\ddot{\psi} + c\dot{\psi} + M(\psi) - k\theta\psi = -mh\ddot{u}_G(t) \tag{7}$$

Which is valid for small displacements *i.e.,* not considering second order geometric effects, which is a valid hypothesis. [Krawinkler, 2004].

Stability coefficient θ in eq. 7 widely used to quantify PΔ effects, relates the hysteretic stiffnesses to the initial elastic stiffness k. For a `sdof` system $\theta = Ph/k$.

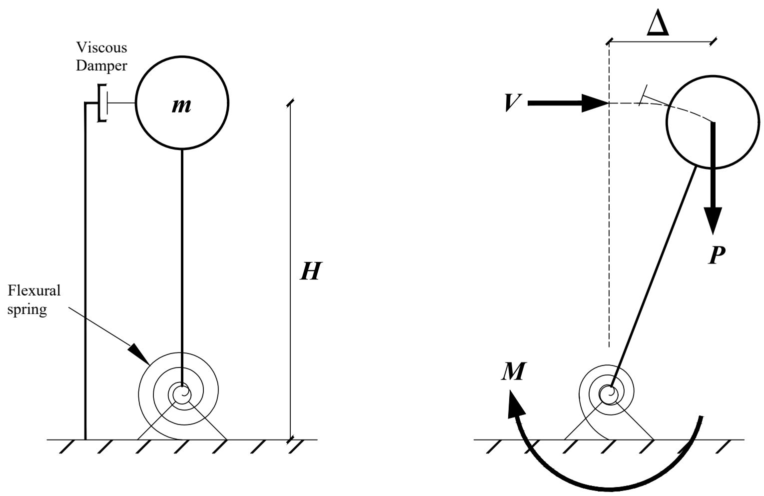

Figure 6: `sdof` damped oscillator subjected to PΔ effects

Normalizing 7 with respect to the yield rotation ϕ_y in order to get a more meaningful dimensionless expression:

$$\omega^{-2}\ddot{\mu} + 2\xi\omega^{-1}\dot{\mu} + \frac{M}{M_y} - \theta\mu = -m^2h^2\ddot{u_G}(t)/M_y \tag{8}$$

Where μ is the associated ductility of the system, and ω, ξ are the geometric linearized form of the natural frequency and the critical damping percentage of the system: $\omega = \sqrt{k/h^2m}$, $\xi = c/2\omega h^2m$. Inspecting the preceding equation, the total force resisted by the spring $(M/M_y - \theta\mu)$ is reduced proportionately to θ, this leads to a "shearing" of the normalized force-displacement relation (fig. 7). If this coefficient exceeds the hardening coefficient α *i.e.,* $(\alpha - \theta < 0)$ the post-yield stiffness becomes negative. In such a condition, the structure may approach dynamic instability during intense excitation. [Bernal, 1998]

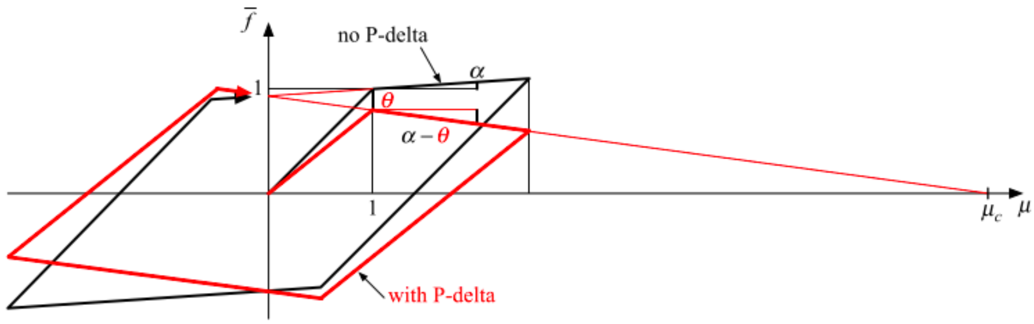

Figure 7: Structural behaviour of an inelastic `sdof` system with and without PΔ [Adam and Jäger, 2012a]

From the figure; it can be seen that $P\Delta$ geometrically transforms the hysteretic behaviour curve, wherein $\alpha - \theta$ governs the collapse capacity of the system, and the system's ductility upper bound is equal to the static collapse ductility μ_c.

In actual structures, such effects are hardly transcendental in the elastic stage of behaviour, however, this may differ in the inelastic stage, wherein the stiffness may be altered significantly by structural damage. Past studies have shown that $P\Delta$ vulnerable frames' behaviour is governed by the first mode [Adam and Jäger, 2011b]. Furthermore, these effects govern the sidesway collapse of certain type of flexible frames [Ibarra and Krawinkler, 2005]. Since `sws` are prone to collapse in such a manner, it would seem valid at first, to suppose that the collapse of a frame that develops a `sws` mechanism governed by the same fundamental mode may be approximated by a bilinear `sdof` system whose properties are defined from a static nonlinear analysis. The aforementioned assumption is based on the fact that the hinging mechanism acts more strongly on the first floor, remaining approximately uniform along the height of the structure. These hypotheses and suppositions do possess some caveats which exceed the scope of this work, the reader is referred to [López, 2015], [Medina and Krawinkler, 2003].

3.2 Influence of the cyclic degradation of elements

Since damage is almost inevitable during strong shaking, appropriate modelling of elements is paramount in order to capture most realistically the true performance of a structure, past research [Ibarra and Krawinkler, 2005]

, [Adam and Jäger, 2012a], has shown that the collapse capacity of a framed structure is a function of the damage state, and is more significant in flexible high-rise structures. As such, it is convenient to study how damage develops in a reinforced concrete element.

Softening —or degradation— in reinforced concrete is caused by rebar slippage, cracking, crushing, rebar fracture or buckling and concentrated plasticity phenomena [FEMA, 2009]. Elements exhibit complex nonlinear behaviour when subjected to load reversals that strain the material beyond its elastic limit, such load reversals are characteristic of intense seisms. Because of this, modelling involves applying appropriate hysteresis rules to structural members in order to accurately capture its behaviour. Hysteresis is defined as a phenomenon in which two or more physical properties respond in a way that is related to their previous common behaviour [FIB, 2003]. Two main hysteretic rules for modelling `rc` can be employed in order to better assess collapse capacity:

- Cyclic degradation
- In-cycle degradation

The fundamental difference of these rules lies on the way strength loss occurs, cyclic degradation does not show this loss during the loading period, while in-cycle does. Pictorially:

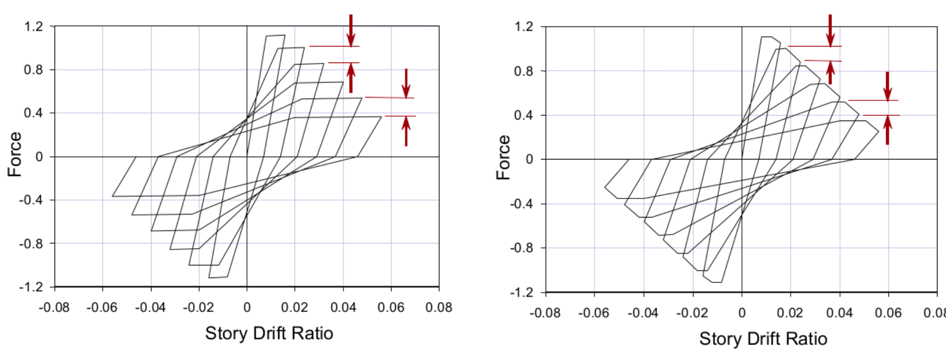

Figure 8: Cyclic and in-cycle strength degradation [FEMA, 2009]

Several hysteretic models for reinforce concrete elements that can incorporate such characteristics have been developed in order to better assess collapse capacity of framed buildings, one of which is the Ibarra, Medina and Krawinkler (`IMK`) hysteresis model for beam and column elements

[Ibarra and Krawinkler, 2005], which is callibrated in such a way as to reduce systemic errors and to take into account uncertainties in the construction process (255 tests were used in the study [Haselton et al., 2009]. This model requires seven parameters which are a function of the geometrical and physical properties of the elements:

$$(M_y, \theta_y, \frac{M_c}{M_y}, \theta_{cap,pl}, \theta_{pc}, \lambda, c)$$

The backbone of this model includes:

- strength deterioration of the inelastic strain hardening branch,
- strength deterioration of the post-peak softening branch,
- accelerated reloading stiffness deterioration,
- unloading stiffness deterioration.

The parameters λ, c control the rate of cyclic deterioration. Cyclic deterioration and normalized energy dissipation capacity describe how the rate of cyclic deterioration changes with accumulated damage. An important aspect is the post-peak negative slope, which models strain softening behaviour crushing, rebar buckling, fracture and bond failure.

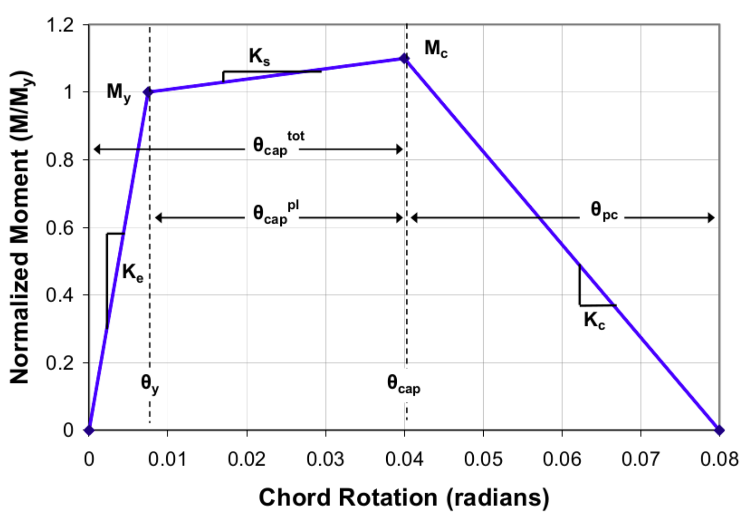

Figure 9: Hysteretic backbone curve according to the `IMK` model for reinforced concrete

This model is implemented in the structural analysis program OpenSees [McKenna et al., 2004], and was specifically developed for the simulation of nonlinear degradation of concentrated plastic hinges for assessing global sidesway collapse. Since several parameters are required to define the backbone of our model, it is convenient to use simplified callibrated expressions that relate geometrical and material properties to hysteretic parameters, research to obtain these expressions have been carried out extensively by Berry and Eberhard [Berry and Eberhard, 2003] and Fardis *et al* [Panagiotakos and Fardis, 2001] with 301 and 1802 tests respectively, further work to incorporate models into software and calibrate parameters to match the model's specifications is still needed.

The following table provides brief information on the chosen model's parameters: [Deierlein and Haselton, 2005].

Table 6: Parameter descriptions of the IMK model [Ibarra and Krawinkler, 2005]

Parameter	Notation	Derivation and hypotheses	Author
Rebar yield	M_y	Whitney block	Fardis
Chord rotation at yield	θ_y	function of geometry	Fardis
Onset of the chord rotation leading to str. loss	θ_{cap}	buckling, stirrup fracture	Fardis
Slope hardening stiffness	K_s	steel hardening, non-linearity of concrete, fibre moment curvature, plastic hinge	Haselton 2006
Post-cap stiffness	K_c	buckling of rebar, stiffness loss due concrete confinement loss	Haselton, Peer 2005
Normalized hysteretic energy dissipation coefficient	λ	progressive cycles of crushing, buckling and longitudinal fracture	Haselton, Peer 2005

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Parameter	Notation	Derivation and hypotheses	Author
Exponent term for λ	c	quantifies the damage state	Haselton, Peer Ibarra 2003

3.3 Assessment of sidesway collapse in framed structures

Proper assessment of collapse is still under research, as such, several authors propose different methodologies for its identification [Haselton and Deierlein, 2007].

- A single or multiple components exceed certain threshold *ie.* ductility demand, linear strain or plastic rotation. This often only indicates "near collapse" behaviour.

- The structure becomes dynamically unstable under transient analysis.

The study uses the latter definition for the assessment of collapse. According to several authors [Vamvatsikos, 2004], [Fragiadakis et al., 2006]; the probability of exceeding a specified engineering demand parameter (`edp`), can be accurately obtained via a technique called Incremental Dynamic Analysis (`IDA`) [Vamvatsikos and Cornell, 2002]. This consists on subjecting the system to a set of scaled intensity measures (`im`) of earthquake motions, obtaining the corresponding levels of structural demand. Afterwards, a statistical analysis is performed on the results, wherein the probability of exceeding said `edp` given a seismic demand level can be computed. Although simple in concept, record to record variability and epistemic uncertainties need to be dealt with appropriately in order to derive a meaningful result. [Vamvatsikos et al., 2015]

Using our definition of a dynamically instability; this type of condition may be identified in an `IDA` curve via a "softening" or "flattening" portion, which correspond to a disproportionate displacement given a small change in the variation of the associated intensity measure.

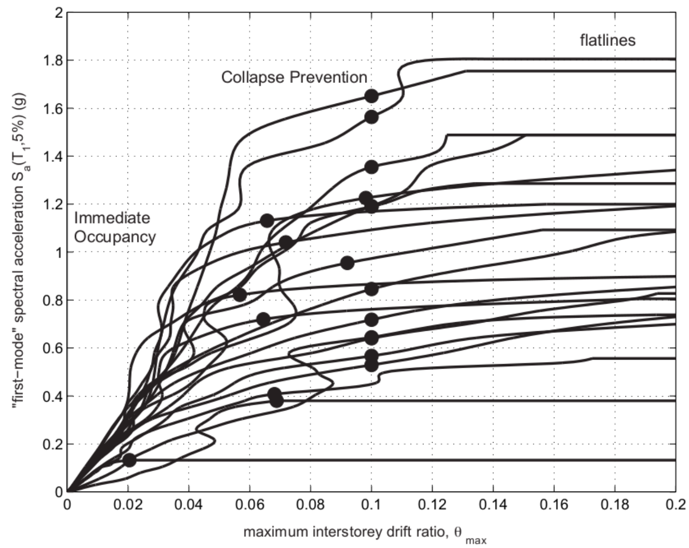

Figure 10: Example of IDA curves for a set of records, showing the flattening portion of the curve indicative of dynamic instability [Vamvatsikos, 2004]

The performance of a system as a function of the probability of exceedance of an edp, for a given set of records can be used to more accurately estimate the compliance of a limit state or its collapse capacity.

As an alternative to IDA, Adam & Jäger [Adam and Jäger, 2012b], have utilized the so-called collapse capacity spectra for systems that possess a $P\Delta$ induced negative post-yield stiffness. These spectra define the intensity for which dynamic instability occurs, given the axial load influence and other system parameters that are relatively easy to quantify. Afterwards, an estimate of the collapse capacity (CC) can be read directly from such a spectra. Hence, they provide a useful tool to quickly and reliably assess the collapse capacity of flexible planar frame structures in a preliminary design process.

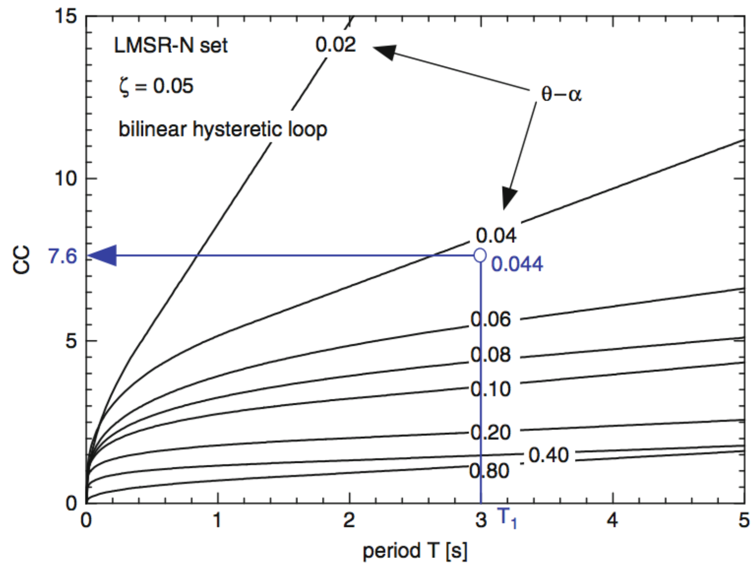

Figure 11: Application of a collapse-capacity spectra to an equivalent non-deteriorating `sdof` [Adam and Jäger, 2012a]

4 Conclusions

Correct performance assessment is a necessary task in order to develop effective design methods or enhance current ones, however, they should explicitly include second order geometric effects ($P\Delta$) since it has been proven that they greatly enhance ductility demands and play a large role in their performance. Current provisions are lacking in this regard, thus, they are not completely effective in providing uniform performance or guaranteeing compliance of the `ULS`. It is evident that more research on the topic and on the development of design methodologies is needed, particularly ones that incorporate the influence of infill walls and the cyclic degradation of elements, as they play a major role on the overall global stability evolution and the performance of the system.

The codes are similar in their treatment of the `sws` condition, having a general trend of increasing design base shear and limiting the maximum admissible height for this type of vertical irregularity. The influence of P-Delta effects in design is addressed via a static stability coefficient that is somewhat well bounded. As expected, seismic behaviour coefficients for design base shear vary significantly from code to code, as do other seismic behaviour factors. What is somewhat remarkable however, is the lack of general agreement on the type of allowed analyses for design and the interstorey strength of stiffness disparity that would lead to a `sws` consideration. This, coupled with different criteria regarding the ULS interstorey drift, suggest at best, that collapse prevention has not been fully solved.

Recommendations akin to those found in the aforementioned codes, continue the force-based design trend, and were in place when high intensity seisms over the world (México '85, Northridge '94, Izmit '99) caused many `sws` to suffer severe damage or to collapse. In a general sense, codes have evolved and have been significantly bettered, however, even though design recommendations and seismic behaviour factors have varied over the preceding years, they still refrain from adressing the collapse-mechanism directly. Strict guidelines intended to combat this pathogenic effect *eg.* `scwb` criterion, have not reached full consensus (as can be deduced from the table). This is partly due to the inherent complexity of the problem, wherein phenomena that are difficult to model such as geometric second order effects and material nonlinearity rule the dynamic behaviour and the stability evolution of the system. Such phenomena, among others, lead to a redistribution of internal forces that changes with time as well as with structural damage, which are out of the scope of static elastic stability coefficients, or other seismic behaviour factors for that matter, which makes them unable to pinpoint the apparition or location of unwanted structural behaviour.

4.1 Open questions

- Influence of infill walls nonlinearity on the global stability evolution and the collapse capacity of `sws` buildings,
- effects of soil-structure interaction on the global performance of `sws` in high intensity earthquakes.

4.2 Future studies

- Evaluate analytically the effects of design recommendations on the collapse capacities of `sws` buildings via `IDA` of models designed with and without design base shear correction,
- develop a design methodology that incorporates $P\Delta$ induced dynamic instability and damage control directly, such that it guarantees uniform performance and `ULS` compliance.

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