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EVALUATION OF SEISMIC ACTIONS IN EGYPTIAN LOADING STANDARDS COMPARED TO INTERNATIONAL CODES

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ABSTRACT

The earthquake resistant design regulations in Egypt have undergone major changes over the past two decades (especially after the October 12, 1992 earthquake), which promotes the need for reconsidering the seismicity of Egypt and the necessity of including seismic loads in the design process. In this work, the seismic load provisions in existing Egyptian loading standards are explained and qualitatively compared to their counterparts in three of the well-known international model building codes. Four numerical case studies are presented to evaluate the seismic actions obtained using seismic load calculation procedures presented in the featured codes. The chosen case studies represent the prevailing low- to mid-rise multistory framed RC building structures used in Egypt. The structural analysis procedure is carried out using both the Equivalent Static Analysis and the Response Spectrum Analysis methods in addition to Linear Time-History Analysis using a set of simulated and natural records for normal patterns of major earthquakes encountered in Egypt. Finally, the results are compared and assessed.

Introduction

Several Egyptian codes of practice have been published in Egypt during the past two decades. The seismic load provisions of these codes are of different design basis and parameters which lead to numerous inconsistencies and omissions associated with these codes. Hence, this work aims to assess the seismic load provisions in existing Egyptian codes of practice in comparison with modern international seismic design building codes, best practices and advances in earthquake-resistant design field. Codes reviewed are the Egyptian code for loads and forces, ECLF 2003 (ECLF201 2003) as well as its predecessor, ECLF 1993 (ECLF201 1993), in addition to the regulations of the Egyptian Society for Earthquake Engineering, ESEE1988 (Sobaih 1988), the European standard, EC8:2003 (CEN/TC250 2003), the International Building Code, IBC 2003 (ICC 2003), and the International Standard, ISO 3010:2001 (ISO/TC98 2001). The paper is organized such that the six codes are first qualitatively compared using the method of cross reference for the terms of (1) design objectives, (2) design level ground motion components and followed by a quantitative comparison conducted for the terms of base shear forces specified by each code and their distributions using the Equivalent Static Force Method (ESFM), and the Response Spectrum Method (RSPM), in addition to Linear Time-History Method (LTHM) to show the level of matching between real recorded earthquake events in Egypt and expected earthquakes intensity stated by Egyptian codes.

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Qualitative Comparison

Design Objectives

Since achieving complete protection against all earthquakes is not economically feasible for most types of structures, seismic building codes result in "earthquake-resistant" buildings, but not "earthquake-proof" buildings. Hence, the design objectives of such codes intend to protect people inside buildings in accordance with the following design requirements: (1) "preventing" collapse at ultimate limit states (no collapse requirement) and (2) "limiting" structural damage at serviceability level limit states (no damage requirement). In the light of the above discussion, Table 1 presents a detailed comparison of the definition of "Design Earthquake" as related to the "Design Requirements" in the six considered codes.

Table 1. Comparison of codes definition of design earthquake as related to the design objectives.

Code	Definition of Design Earthquake
ESEE 1988	Descriptive definition is given. Although inexplicitly stated, ESEE 1988 seismic zoning map, defining the design earthquake, conforms to the hazard level of Map No.2 of the set of maps developed by (Ahmed et al. 1992) and (Sobaih et al. 1992), having 10% probability of exceedance in 100 years (corresponding to return period of 500 years) for RC buildings associated with reliability importance factor of 1.0. In addition, the new set of maps, published in 1992, is recommended by ESEE to replace the original regulation map.
ECLF 1993	Only descriptive definition is given using words such as "Severe", "Moderate", and "Weak" Earthquake, but no specific probability of exceedance limit is specified.
ECLF 2003	Same definition presented in EC 8:2003.
EC 8:2003	**No Collapse Requirement:** The design seismic action is expressed in terms of: a) The reference seismic action associated with 10% probability of exceedance in 50 years (corresponding to return period of 457 years). b) The importance factor, γ_I, to take into account reliability differentiation. The different levels of reliability are obtained by multiplying the reference seismic action or, when using linear analysis, the corresponding action effects by this importance factor **No Damage Requirement:** The seismic action to be taken into account for the "damage limitation requirement" has a probability of exceedance, 10% in 10 years (corresponding to return period of 95 years).
IBC 2003	**Total Requirements:** IBC 2003 does not relate the objectives of seismic design to the seismic fortification levels. Instead, it states total requirements to limit damage and prevent collapse. Hence, it defines the earthquake with 2% percent probability of exceedance in 50 years as the maximum considered earthquake (MCE), and its return period is 2005 years. The Design earthquake of IBC 2003 is two-thirds of the MCE. **Reliability Differentiation:** incorporated by the Occupancy Importance Factor, I_E, based on the structure use group.
ISO 3010:2001	Both "No Collapse" and "No Damage" Requirements for different reliability levels are presented in descriptive fashion using words such as "Severe", and "Moderate" but no specific probability of exceedance limit is recommended by the International Standard.

Design Level Ground Motion

The design earthquake ground motion characteristics is typically presented by seismic design codes in terms of three components: (1) the elastic response of the basement rock (usually as acceleration spectra), (2) the relative seismicity at the site (commonly presented as a suite of zonation maps), (3) a modification function which is applied to the motion at bedrock beneath the site to allow for near surface soil conditions (presented as either a simple amplification factor or as a more complex soil property related function). For the considered codes, the elastic design response spectra are compared in Fig. 1, and the design input parameters related to relative seismicity and subsoil amplification effects are compared in Tables 2 and 3, respectively.

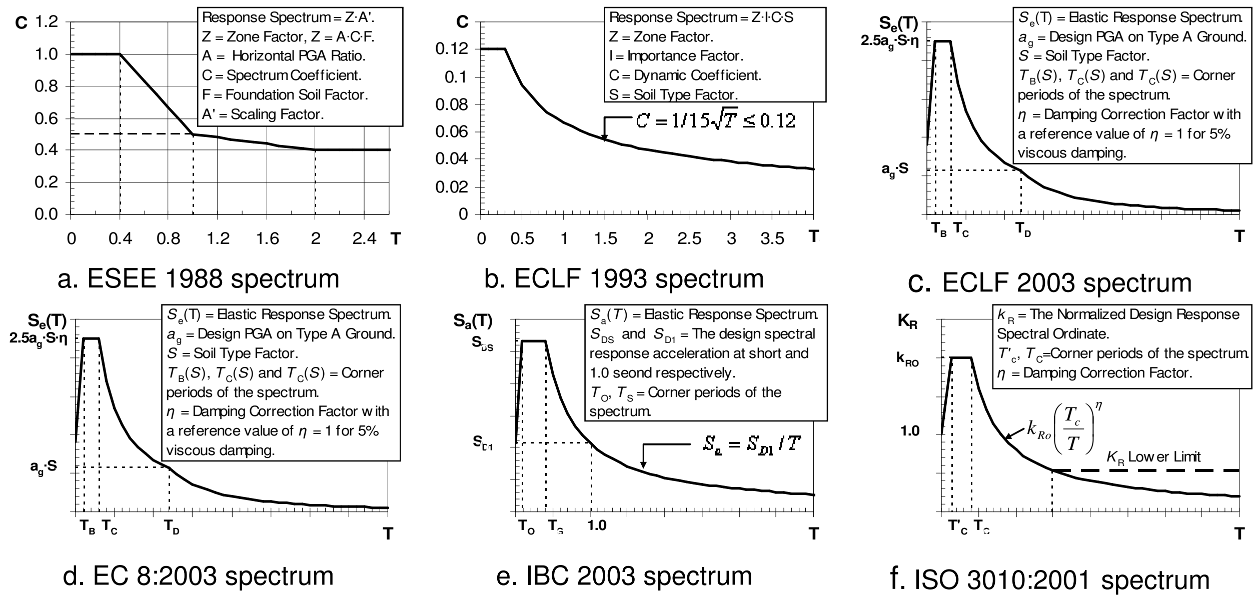


Figure 1. Comparison of codes elastic response spectra.

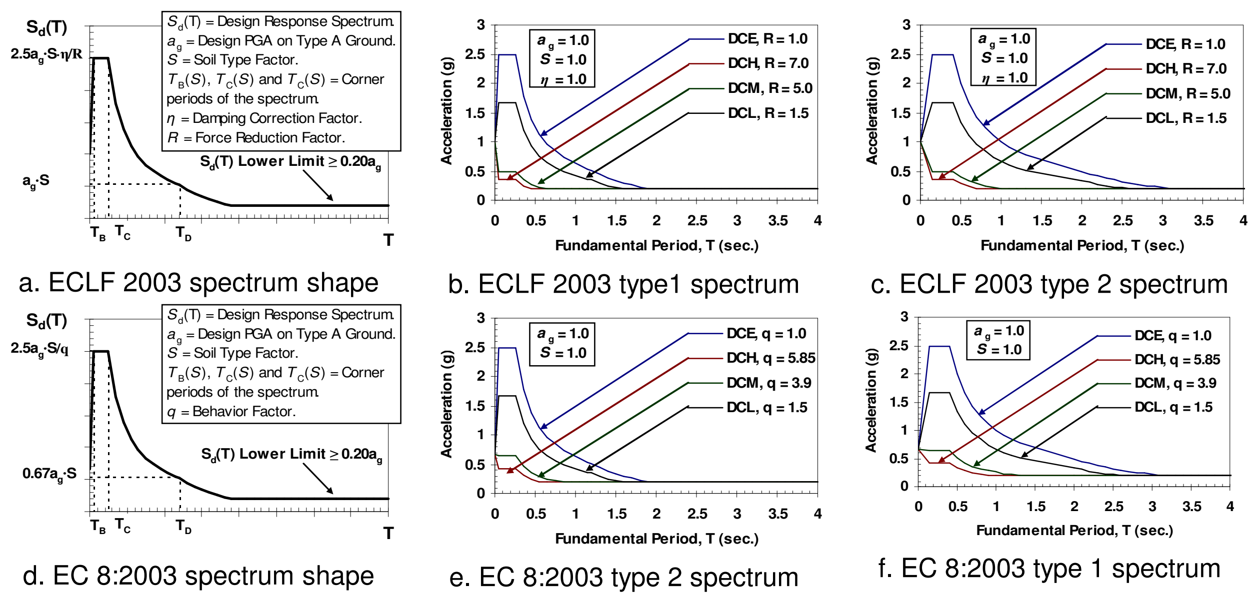


Figure 2. Comparison of ECLF 2003 and EC 8:2003 ductility modified response spectra.

The comparison shows that ECLF 1993 spectrum is presented as a simple uniform coefficient followed by an exponential decay. ECLF 2003 adopts the same response spectrum shape proposed by EC 8:2003. The current code spectrum conforms to the modern trend, indicated by other International Standards considered in this study. This is to publish the design spectra in a general parametric form where the ordinates of each parameter and the characteristics of the curve between them are functions of Damping Ratio, Design PGA, and Site Subsoil Class. Both ECLF 2003 and EC 8:2003, have introduced two types of spectral curves. These are the “normal” spectrum shape (see Figs 2.b, and 2.e) with low amplification in the long period range, and the “long period-rich” spectrum (see Figs 2.c, and 2.d). In addition, both codes introduced a ductility modified spectra obtained by introducing a ductility related force reduction factor into the formation of the elastic response spectrum to avoid explicit inelastic analysis. These spectral curves are plotted in Fig. 2 for different ductility classes specified by each code for the case of moment-resisting space frame system. IBC 2003 define the seismic hazard and consequently the design spectrum in different manner where the hazard is described in terms of two spectral accelerations taken at short periods (0.2 seconds) and at 1-second period instead of one single value representing the design PGA.

Table 2. Comparison of codes relative seismicity design input parameters.

Code	**Relative Seismicity Design Input Parameters**
ESEE 1988	Contour map with 4 seismic zones describing the seismic hazard in terms of (1) horizontal PGA ratio, A and (2) Macroseismic Intensity Scale grades.
ESEE 1993	Contour map with 3 seismic zones describing the seismic hazard in terms of the Zone Factor, Z.
ECLF 2003	Contour map with 5 seismic zones describing the seismic hazard in terms of the Design PGA on Type A Ground (i.e., Rock), a_g.
EC 8:2003	Requires seismic contour maps to express the seismic hazard in terms of the Reference PGA on Type A Ground (i.e., Rock), a_{gR}.
IBC 2003	Provides 2 sets of contour maps, based on contours of 5% damped elastic spectral accelerations for periods of vibration of 0.2, and 1.0 seconds, respectively
ISO 3010:2001	Requires seismic contour maps to express the seismic hazard in terms of either Zoning Factor, K_z, or Representative values of earthquake ground motion PGA intensity, $K_{E,u}$ or $K_{E,s}$.

Table 3. Comparison of codes subsoil amplification design input parameters.

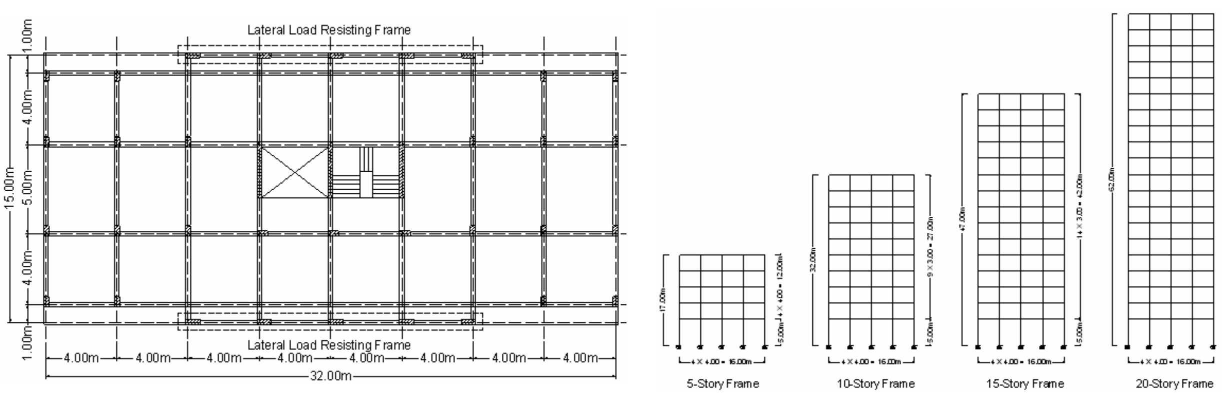
Code	**Subsoil Amplification Design Input Parameters**
ESEE 1988	• F = Foundation Soil Factor: The design spectrum is adjusted for site class effects by defining the Zone Factor, $Z = A \cdot C \cdot F$ where the coefficient C is given in Fig.1.a. • F is determined by descriptive subsoil classification scheme composed of 3 soil profiles (Rock and stiff soils, Medium density soil, and loose or soft soil).
ESEE 1993	• S = Soil Factor: S is used as one of the factors defining the base shear coefficient. • S is determined by descriptive subsoil classification scheme composed of 3 soil profiles (Rock and stiff soils, Medium density soil, and loose or soft soil).
ECLF 2003	Same provisions presented in EC 8:2003 except that only 4 subsoil categories (A to D) are given.
EC 8:2003	• S = Soil Factor: S, and the corner periods: $T_B(S)$, $T_C(S)$ and $T_D(S)$ define the shape of the design response spectrum depending upon the site class. • S is determined by subsoil classification scheme of 5 categories (A to E) plus two special ground types (S_1 and S_2) based on 3 soil parameters: (1) Average shear wave velocity, (2) Standard penetration test blow-count, (3) Undrained shear strength of soil.
IBC 2003	• F_a & F_v = Site Coefficients: The design earthquake accelerations at short and 1-second periods, S_s and S_1, are adjusted for site class effects as follows: $S_{MS} = F_a \cdot S$ & $S_{M1} = F_v \cdot S_1$ then the design values are taken as two-thirds of S_{MS} and S_{M1}values. • F_a & F_v are determined by classification scheme of 6 categories (A to F) based on 3 soil parameters: (1) The average shear wave velocity, (2) Standard penetration Test blow-count, (3) Undrained shear strength of soil.
ISO 3010:2001	• Soil effect is present implicitly in the formation of the Normalized Design Response Spectrum by defining the constant acceleration range value, k_{Ro}, and the two corner periods T_c and T'_c. The following general descriptive categories of soil conditions are mentioned in the standard normative annexes: (1) Stiff and Hard Soil, (2). Intermediate Soil, (3). Loose or Soft Soil. But not specific measures are presented and proper geotechnical studies are recommended.

Quantitative Comparison

Description of Example Building Structures

The four example building structures, with the typical floor plan shown in Fig. 3.a and varying elevation heights of 5, 10, 15 and 20 stories respectively, are chosen to represent the common layouts of low- to mid-rise multistory framed RC building structures in Egypt. The examples are residential buildings located in Cairo City. For comparison purpose, the buildings are assumed to be founded on rock ground conditions with foundation subsoil factor equal to unity. This is to eliminate the effect of different codes classifications of subsoil conditions on the calculated base shear force. In addition, the buildings have regular configurations in both plan and elevation. Hence, no actual torsional eccentricity exists and only design accidental torsional eccentricities, specified by each compared code, shall be considered in the comparison. A common characteristic cube strength of 250 kg/cm^2 for concrete and yield strength of 3600 kg/cm^2 for steel are utilized in the design. The buildings are dimensioned and detailed in accordance with the Egyptian code for design and construction of RC structures (ECCS203 2001). More information regarding, member cross-section sizes and reinforcement details are given elsewhere in (Khattab 2003).

For the sake of comparison purpose, only the seismic lateral forces in the longer plan direction (X-direction) are considered in this study where the lateral loads are resisted by moment-resisting frame system. Only the two perimeter frames, surrounded by the dotted lines in Fig. 3.a and depicted in Fig. 3.b, are utilized to carry the whole lateral forces in this direction in addition to their share of the vertical loads, since their location on the perimeter provides more torsional resistance. Refined 2D analytical models, developed using SAP2000 (Habibullah and Wilson 2005), are utilized in the dynamic analysis of the lateral loads resisting frames. An integrated set of computer spreadsheets have been developed to automate the process of calculating the equivalent static analysis base shear forces and their distributions on the studied frames. In addition, SAP2000 supports transferring data from/to computer spread sheets. Hence, this feature was utilized to fully automate the whole analysis procedure.



a. Typical floor plan layout   b. Lateral loads resisting frames in X-direction

Figure 3. Example building structures.

Procedures of Seismic Analysis

Three sets of comparison cases are conducted in this study. In the first set, each code is applied using its own zonation map (i.e., Zone Factor, Z = 0.20 in ECLF 1993, and PGA = 0.15 in ECLF 2003, and 0.04 in ESEE 1988). In the other two sets, all codes are applied using unified PGA of 0.15 and 0.04g, respectively. In each comparison set the structures are analyzed by ESFM considering four different levels of ductility classes. The first three levels conform to the common practice of modern design codes that classify buildings seismic performance into three ductility classes. These namely are High (DCH), Medium (DCM), and Low (DCL). The fourth level assumes complete Elastic response of the structure (DCE). The later case is introduced, for comparison purpose only, to eliminate the effect of using codes differently calibrated values for the Force Reduction Factor, R, on the obtained base shear forces. It should be noted that some of the compared codes such as ESEE 1988 and ECLF 1993 provide only two levels of ductility classes upon which the factor R is evaluated. These namely are "Ductile" and "Limited Ductility" (or Non-ductile) Moment-Resisting Frame Systems. Hence, the factor R is chosen for such codes assuming both DCH and DCM correspond to "Ductile" class, while DCL corresponds to "Limited Ductility" class. The RSPM analysis is carried out for each structure by applying each of the six codes elastic response spectrum scaled to PGA of 0.15 and 0.04, respectively. In addition, a RSPM analysis case is carried out for each structure considering PGA of 0.20 for ECLF 1993 analysis cases.

Since the results of the LTHM depend mainly on the characteristics of the used acceleration time-history records and the shapes of their corresponding elastic response spectra, the soul aim of using this method here is to assess the results obtained by other codes specific analysis procedures (i.e., ESFM and RSPM) against a set of time-history records which represent the common characteristics of earthquake events encountered in Egypt main land and neighboring seismic active areas lying on the Mediterranean Sea and the Read Sea regions. Considering the distribution of earthquake hazard sources in Egypt, the six acceleration time-history records, given in Table. 4, are selected. Each of the six records is applied three times to each example building structure with PGA scaled to 0.20g, 0.15g, and 0.04g, respectively. The time-history analysis is performed using the modal superposition technique considering a time step of 0.005 seconds and a constant damping of 0.05.

Table 4. Characteristics of acceleration time-history records.

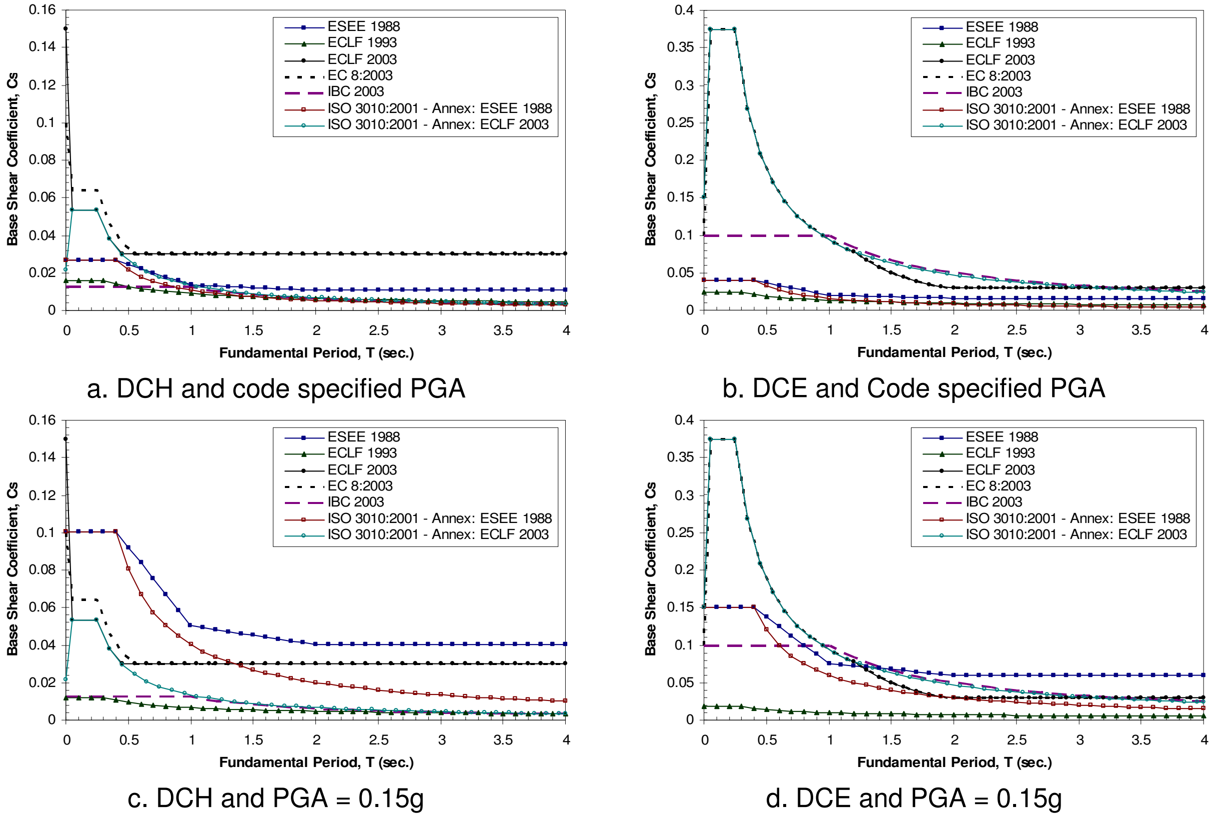
Record Reference	Earthquake Name	Station	M	PGA (g)	Local Geology
AQABA-EIL[a]	Gulf of Aqaba 22/11/1995	Eilat	7.1 M_w	0.091	Stiff Soil
AQABA-HAD[a]	Gulf of Aqaba 22/11/1995	Hadera Meor David	7.1 M_w	0.019	Stiff Soil
BENI-SUEF-SIM[b]	Dahshour(Cairo) 11/10/1999	Koreimat (Simulation)	5.0 M_b	0.008	Rock
DAHSHOUR-SIM[b]	Beni Suef 12/10/1992	Koreimat (Simulation)	5.9 M_b	0.002	Rock
DINAR-DIN[a]	Dinar, Turkey 1/10/1995	Dinar-Meteoroloji Mudurlugu	6.4 M_w	0.319	Soft Soil
KOZANI-KOZ[a]	Kozani, Greece 13/5/1995	Kozani-Prefecture	6.5 M_w	0.207	Rock

[a]Obtained from European Strong Motion Database (ESD), (Ambraseys et al. 2002).
[b]Obtained from Egyptian National Seismic Network (ENSN), (El-Sayed 2006).

Analytical Results

Equivalent Static Analysis Base Shear Forces

In the first comparison case, each code is applied using its own seismic zonation map. Hence, the influence of different design PGAs among the three Egyptian codes is very clear on the resulting forces, especially in the analysis cases considering complete elastic response (DCE) since the influence of force reduction factors is removed and the only governing factor is the applied design PGA. Therefore, this comparison case is useful to assess the seismic vulnerability of existing structures in Egypt designed to different seismic provisions and seismic zonation maps. In the second and third comparison cases, all codes are applied using unified design PGA of 0.15g and 0.04g, respectively, hence considered fair comparisons from which the level of codes conservatism can be judged as presented in the following paragraphs. Fig. 4 shows some of the typical examples obtained for ESFM base shear coefficient spectra (i.e., Base Shear Coefficient (base shear/building total weight), C_s, versus the Fundamental Period, T).



a. DCH and code specified PGA

b. DCE and Code specified PGA

c. DCH and PGA = 0.15g

d. DCE and PGA = 0.15g

Figure 4. ESFM base shear coefficient spectra (base shear force/total building weight vs. period).

ECLF 2003 results come as expected confirming to those of its reference code, EC 8:2003, since they both adopt the same procedure except that ECLF 2003 forces are lower than those of EC 8:2003 due to the higher force reduction factors used in the Egyptian code. They are unjustified due to the lack of corresponding reinforcement detailing requirements in the current Egyptian code of practice for RC structures, ECCS 2001 (ECCS203 2004) where such requirements are necessary to ensure reaching the assumed design ductility level by the loads code. It should be noted that in the long period range the base shear coefficient spectrum curves of both codes coincide together since both codes use a spectral acceleration lower limit equal to 0.2 of the design PGA.

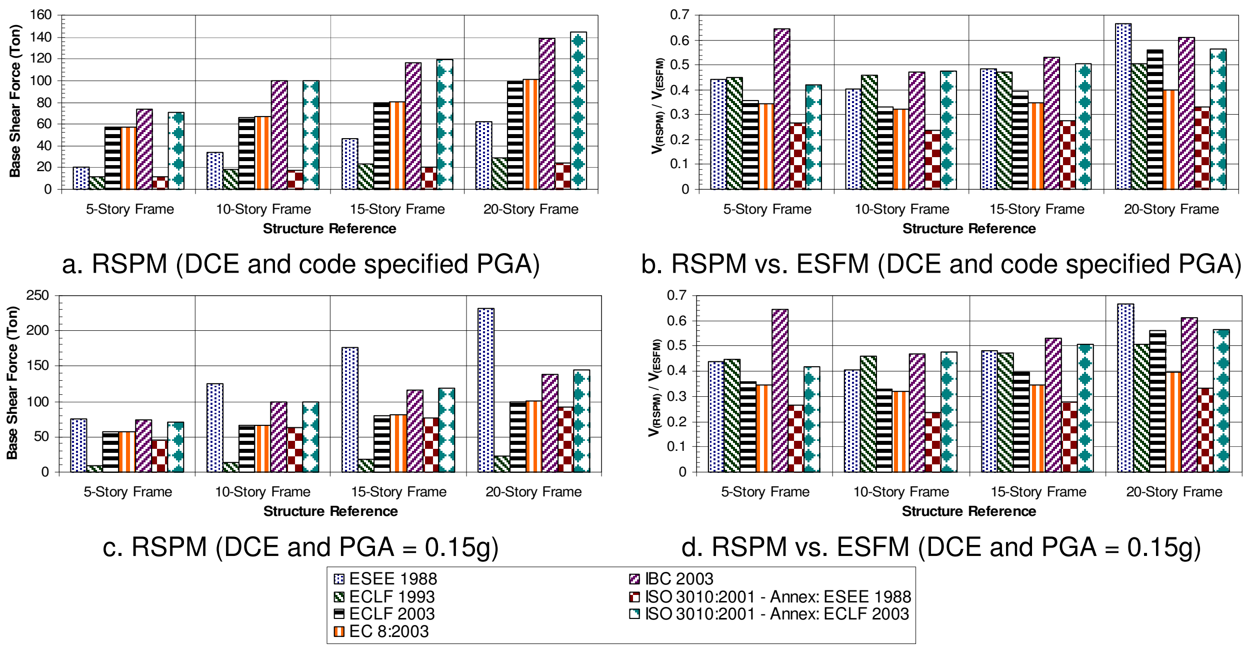
ECLF 1993 gives the lowest values, obtained from all analysis cases, compared to other codes. This can be traced back to the following reasons: (1) ECLF 1993 considers only the building dead loads in calculating the base shear forces for buildings with floor live load < 500 kg/m^2. In the same time, the code states much lower limits for minimum allowable floor live loads in most types of buildings, in our case 200 kg/m^2 for residential buildings, (2) ECLF 1993 express the expected earthquake intensity in terms of seismic zone factor instead of explicit design PGA which makes the resulting forces insensitive to the change in the design PGA and be tied to the numerical values supplied by its seismic zonation map.

ESEE 1988 regulations showed conservative results in analysis cases considering high and medium ductility. This is due to the fact that ESEE 1988 utilizes very high structural system factors which implicitly lead to much lower force reduction factors than other codes considered in this study. For example, ESEE 1988 utilizes a force reduction factor of approximately 1.493 for DCH compared to 7.0 in ECLF 2003, 5.85 in EC 8:2003 and 8.0 in IBC 2003 which means that ESEE 1988 assumes almost elastic response compared to other codes. This fact is very clear in comparison cases considering low ductility (DCL) or complete elastic response (DCE) where ESEE 1988 showed very reasonable results while ECLF 2003, EC 8:2003 and IBC 2033 showed much higher force values.

Comparing the results of ISO 3010:2001 analysis cases, which use either ESEE 1988 or ECLF 2003 as complementary national annexes, with the corresponding analysis cases which use these codes show that the force values are the same in the constant acceleration range of the spectrum then ISO 3010:2001 force values start gradually to become lower than the corresponding code in the decreasing part of the spectrum. This difference arises form the following reasons: (1) the different shape of the two codes response spectra in this range of periods, (2) the different spectral acceleration lower limit specified by each code (curves given in Fig. 4 are plotted assuming no lower limit of for ISO 3010:2001).

Dynamic Analysis Base Shear Forces

The comparison of codes base shear forces obtained from RSPM analysis cases showed similar trends to those seen in the corresponding ESFM analysis cases. In the first comparison case, the difference between codes is due to the difference in codes PGA used in constructing their response spectra. In the second and third comparison cases, the lowest forces from all analysis cases are obtained by applying ECLF 1993, and ECLF 2003 results conform to those of its reference code, EC 8:2003. Typical examples of the obtained RSPM base shear forces are shown in Figs. 5.a, and 5.c. and the same analysis cases are compared to their ESFM counterparts in Figs. 5.b, and 5.d, respectively.



a. RSPM (DCE and code specified PGA)

b. RSPM vs. ESFM (DCE and code specified PGA)

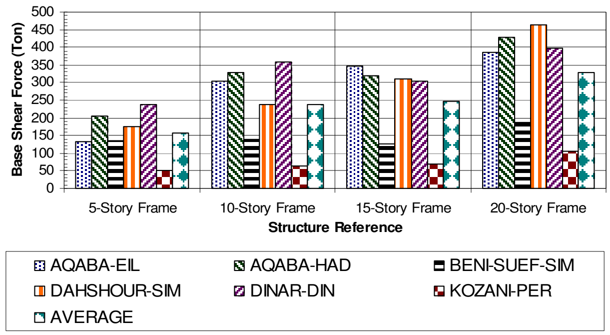
c. RSPM (DCE and PGA = 0.15g)

d. RSPM vs. ESFM (DCE and PGA = 0.15g)

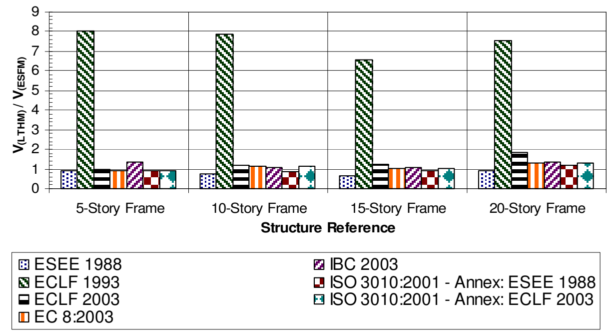
Figure 5. Comparison of codes base shear force ratio between RSPM and ESFM

The comparisons show that RSPM base shear is generally less than the corresponding ESFM base shear. This situation is normally handled in seismic building codes by scaling up the dynamic analysis base shear force to a minimum safe ratio of the corresponding ESFM base shear. ESEE 1988 and ECLF 1993 state this ratio as 0.80 and 0.90, respectively for both RSPM and LTHM. On the contrary, ECLF 2003 does not specify any minimum allowable limit for RSPM base shear. However, it states that LTHM base shear shall not be less than 0.9 of the corresponding RSPM base shear.

Fig. 6 shows example of the typical results obtained from comparing the average of LTHM base shear forces to their ESFM counterparts. The comparisons show that the results of all codes are generally confirming to the dynamic analysis results except ECLF 1993 which resulted in very low forces compared to the LTHM forces. This confirms the fact that it underestimates the ESFM base shear forces. It is also noticed that as the analyzed structure height increases, the ratio between LTHM base shear and the corresponding ECLF 2003 base shear increases, which means that ECLF 2003 tends to underestimate the ESFM base shear forces for higher buildings more than other compared codes.



a. LTHM (DCE and PGA = 0.15g)

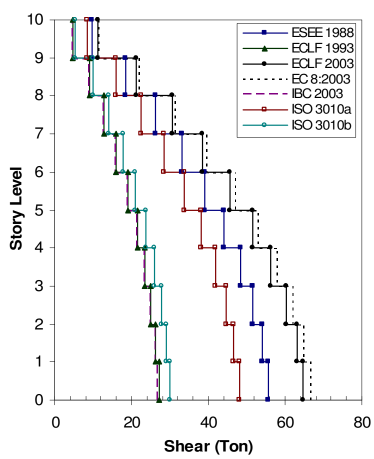


b. LTHM vs. ESFM (DCE and PGA = 0.15g)

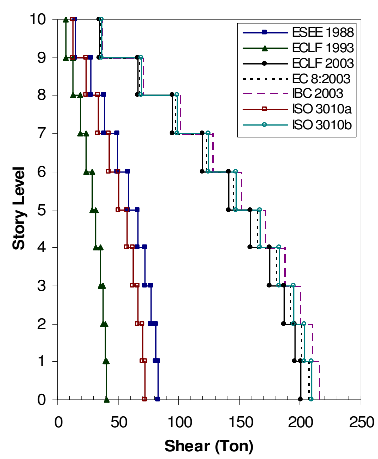
Figure 6. Comparison of codes base shear force ratio between LTHM and ESFM

Distribution of Forces

Comparing the distributions of base shear forces show that no remarkable differences can be noticed in the force distribution patterns of the six considered codes where the difference between any two codes distributions is directly proportional to the difference in their base shear force. The only exception to this trend is found in the top story forces for distributions of 15 and 20 story buildings analyzed using ECLF 1993, ESEE 1988, and ISO3010:2001 with ESEE 1988 national annex since they specify an additional concentrated top force for high rise buildings characterized by long periods. For other buildings analyzed using this code, the concentrated top force is equal to zero, hence their distributions go with the normal trend. Typical example of obtained base shear distribution patterns is shown in Fig. 7 for 10-story frame.



DCH and code specified PGA



DCE and code specified PGA

[a]ISO 3010a ≡ ISO 3010 - Annex: ESEE 1988.
[b]ISO 3010b ≡ ISO 3010 - Annex: ECLF 2003.

Figure 7. Base shear force distribution for 10-story frame in different ESFM analysis cases.

Conclusions

In the current study, the seismic load calculation procedures in existing Egyptian codes of practice have been evaluated both qualitatively and quantitatively in comparison with three of the well-know International codes. It was found that ECLF 2003 provisions are clones of those stated in EC 8:2003. However, ECLF 2003 resulted in lower base shear force levels due to the unjustified force reduction factor values which are higher than their EC 8:2003 counterparts. On the other hand, both ECLF 1993 and ESEE 1988 state very conservative values for the force reduction factor. Hence, it is recommended that these values should be reconsidered in future code editions and should be supplied with sufficient detailing requirements and reinforcement details in future editions of the Egyptian code of practice for RC structures. RSPM results showed similar trends to those seen in the corresponding ESFM analysis cases. However, RSPM base shear were generally found less than the corresponding ESFM base shear which emphasize on the necessity of including a minimum RSPM to ESFM base shear ratio in the current Egyptian code of practice, ECLF 2003. This will make it consistent with other studied codes. ESFM results showed reasonable matching with the LTHM analysis results except with ECLF 1993 which completely underestimated the ESFM base shear force in all analysis cases compared to other codes. No remarkable differences were noticed in the force distribution patterns of the six considered codes where the difference between any two codes distributions is directly proportional to the difference in their base shear force. Finally, due to the inconsistencies found among Egyptian codes, the seismic vulnerability of existing buildings designed to older codes should be reevaluated. In addition, codes harmonization toward a unified definition of the seismic hazard and its related design input parameters should be thought of as a necessity in future loading standards.

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