INTRODUCTION TO IBC SEISMIC FORCES

1.0 INTRODUCTION

This presentation discusses seismic forces mainly from the perspective of the core requirements of the ICC's IBC 2009. The IBC 2009 caters only for new structures, and is based on the model code NEHRP 2003 –" Seismic Provisions for New Buildings and Other Structures". It is linked to the ASCE 7-05 "Minimum Design Loads for Buildings and Other Structures" and when IBC 2009 is referred to in this presentation, the ASCE 7-05 is implied.

The IBC 2009 presents procedures for the calculation of the seismic forces on a building structure, on an element of the building structure, on a non-building structure, and on an item attached to the building or non-building structure.

Particular attention is paid to the use of *equivalent static* procedures that attempt to capture the principal effects of the essentially dynamic scenario. These are:

a. The detailed procedure for the total seismic force on a structure, V
b. The simplified procedure for the total seismic force on a structure, V
c. The seismic forces on components attached to the structure, F_p

The great value of "equivalent statics" is that the subsequent analysis of the structure can proceed using the well-known linear-elastic techniques of structural mechanics thereby avoiding the complexity of dynamic analysis. As will be discussed later, dynamic analysis methods cannot be completely avoided and are necessary for many structures of practical interest.

Approach "a" above is intended for a wide range of applicability. After its presentation, the conditions of its applicability are given. Approach "b", which requires the least calculation effort, is permitted only for buildings of 3 or less stories that are not drift-controlled, along with certain other criteria.

In this presentation, only method "a" is described. Minor discussion of analysis and design issues is also presented. The seismic forces on components attached to the structure are determined using formulae in the IBC, and is beyond the scope of this presentation.

2.0 BACKGROUND

Unlike the gravity load and the wind load (with the exception of slender structures), the seismic load on a structure is generated by the deformation of the structure as it vibrates. The seismic load is therefore part of the dynamics of the structure and is governed by

Newton's Second Law. The gravity and effective wind load are governed by Newton's Third Law.

The seismic load is the inertia force on the structure and its magnitude and distribution on the structure continually change in time, t, from the start of the vibration. For design purposes, we are interested in determining the maximum seismic force and its distribution on the structure during the vibration.

As a phenomenon of dynamics, the seismic force therefore depends on the distribution of mass and stiffness throughout the structure. Furthermore, as structures are typically designed to respond inelastically under the vibration, the seismic force also depends on the manner by which plasticity spreads through the structure. These two factors are critical in determining the maximum magnitude and the distribution of the seismic force and must be considered in any approach which attempts to quantify the seismic force.

The dynamics factor must capture the fact that the seismic force represents a magnification or amplification of the vibration of the ground due to a resonance effect of the structure, and that the maximum amplification changes with the free vibration period of the structure. As you may remember, the free vibration period, $T = \sqrt{(M/K)}$ for an elastic structure, so *if the structure is considered as one uniform or regular mass*, the dynamics factor hence maximum seismic force depends on the mass (M), and elastic stiffness (K) of the structure.

Since the seismic force is an inertial force, we can express the force in terms of the acceleration of the structure. The graph of the maximum acceleration as a function of the free vibration or natural period is called the *acceleration response spectrum*.

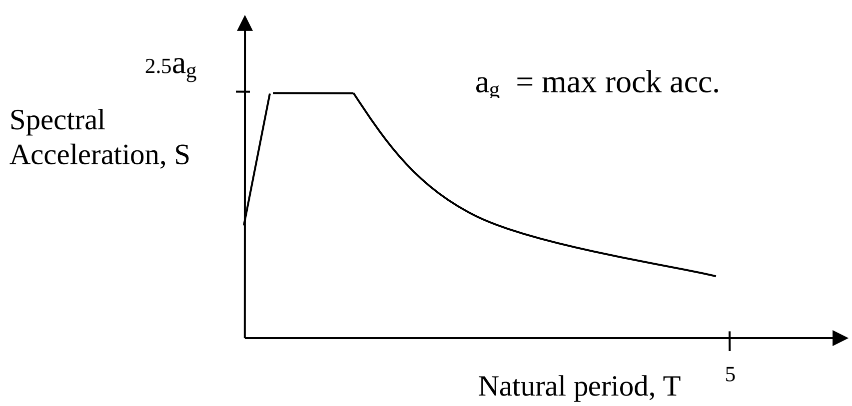


FIG. 1. Acceleration response spectrum

The acceleration response spectrum shown above is a typical graph of the averaged and smoothed maximum acceleration of elastic structures founded on rock, due to an earthquake. Notice that the maximum magnification of the rock acceleration is about 2.5. Notice also that structures with a small period (i.e. high stiffness or shorter structures) experience higher magnification of the ground acceleration, and that after the plateau, the magnification rapidly decreases (i.e. for more flexible or taller structures).

But structures are not typically founded on rock – there is soil between the rock, where the earthquake originates, and the structure. Structures in the higher range of periods and located beyond about 80 to100 km from the epicenter of the earthquake experience additional acceleration due to the resonance of the soil as the ground acceleration becomes more periodic and its period lengthens. It was believed that this soil resonance effect was only significant for these longer period structures. However, it was recently acknowledged that the soil resonance effect also occurs in the short period range, especially on soft sites. It is therefore important to consider these facts by defining the spectrum in terms of two factors - the short period acceleration, and the acceleration at a longer period. By convention, spectra are described at periods of 0.2 second, and 1.0 second. Any spectrum can be graphically constructed given the spectral accelerations at the shorter and longer periods. These are called the S_S and S_1 respectively.

The other important factor – the effect of the spread of plasticity or inelasticity throughout the structure is now considered. As the seismic force increases, a ductile structure yields at one location, then another, then another, and so on, until the structure can no longer sustain the force. In deriving formulae for the seismic force, a uniform or regular ductile frame structure is historically the basic reference for examining the effect of the plastic spread (i.e. succession of hinge formation), on the magnitude of the force.

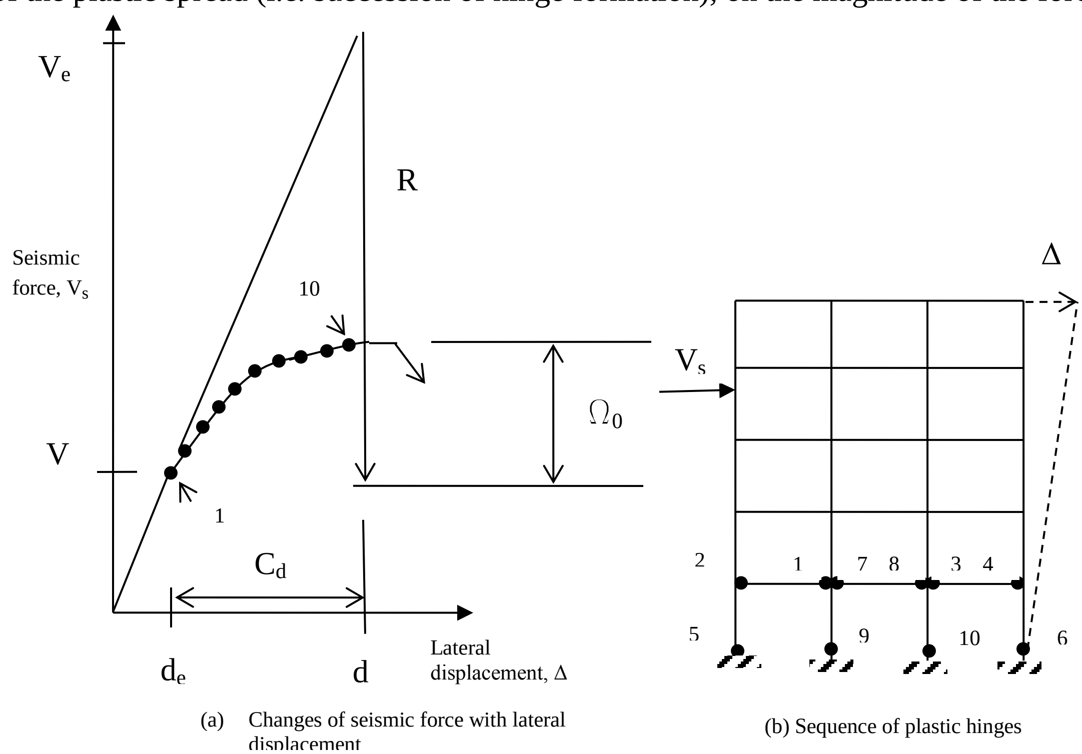


FIG. 2. Changes of the seismic force

Fig. 2(b) shows the sequence of formation of the plastic hinges in an example moment-resisting frame as the resultant seismic force, V_s, is applied. "Δ" is the corresponding lateral displacement of the structure at the top. Though Fig. 2 shows V_s and Δ acting only in one direction, the seismic force and displacement, of course, reverse direction under the vibration.

Note in Fig. 2(a) that V, the seismic force used for design called the *base shear,* is not the maximum seismic force but the seismic force at which the first plastic hinge forms. The reason that V is used for design and not the maximum seismic force is that the gap between V and the maximum force can be used to cater for especially vulnerable members of the structure. The gap between V and the maximum seismic force is represented by the *overstrength factor*, Ω_0. As additional hinges form, the load-displacement curve becomes nonlinear as shown since when a hinge forms the stiffness reduces significantly. As each hinge is formed the structure redistributes stress to other sections until no additional sections can be found. This is when the maximum force or resistance of the structure occurs.

If the structure did not enter the inelastic zone and form hinges but rather remained linear elastic, then the V_s - Δ graph would have been a straight line as shown. If d is the displacement at near-collapse of the structure, then at d the seismic force would have been V_e. The gap between V and V_e is represented by R, the *ductility factor* or *capacity reduction factor,* or *response modification factor*. From Newton's Second Law, V_e = M x S. But S is the spectral acceleration for an elastic structure. Therefore, this means that the effect of inelasticity on the magnitude of the seismic force can be determined just by dividing S by R. This approach to converting the elastic spectrum to an inelastic spectrum is the one used in the U.S seismic codes, but others are possible.

C_d represents the gap between the inelastic displacement d and the elastic displacement d_e.

The above discussion of the magnitude and distribution of the seismic force suggests that this force can be expressed by a formula. This formula is called the *base shear equation* and is the subject of the following sections. The procedure for its use, which includes its distribution to each floor, is called the *equivalent lateral force* (ELF) procedure, or the linear static procedure (LSP).

It must be noted however that the base shear formula gives reliable results only if the structure is not too tall, and is uniform or *regular* vertically in both elevations, and in plan for each floor, in terms of the mass, stiffness, and the strength of its elements. The IBC 2009 however does allow the use of the base shear equation (ELF procedure) for certain mild cases of irregularity, and if the structure is not too tall.

In the case of significant irregularity the ELF procedure will be inaccurate, especially as regards the distribution of the seismic force over the structure. In this case, if the irregularity is with respect to its mass and/or stiffness distribution only, the seismic forces can be determined from the *response spectrum analysis,* also called the linear dynamic procedure (LDP).

If the structure has irregular strength distribution, the IBC 2009, which is for new buildings, requires that the arrangement of the structure and/or its elements be adjusted to remove the strength irregularity. Interestingly, if the building is an existing building and certain other criteria are met, the seismic force on the structure can be determined using the *nonlinear static procedure* (NSP) which involves plastic collapse analysis methods. NSP is not supported in the IBC 2009 for new buildings except if the seismic design is based on supplemental damping.

Otherwise, the seismic forces can be determined via the *nonlinear dynamic procedure* (NDP) in which case the analysis is based directly on the ground vibration, and the inelastic properties of the structure. The LDP, NSP, and NDP procedures are beyond the scope of this presentation.

At the end of this presentation, sketches of the IBC 2009 classification scheme for *irregular structures* are shown. The design of irregular structures in terms of rules for the structural provisions, and the determination of seismic forces on individual elements, is given in the IBC 2009 as a function of the level of seismic risk (called the Seismic Design Category to be discussed in subsequent sections). Seismic risk is in terms of the economic and social consequences of the failure of the structure.

3.0 THE BASE SHEAR EQUATION, V – DETAILED PROCEDURE

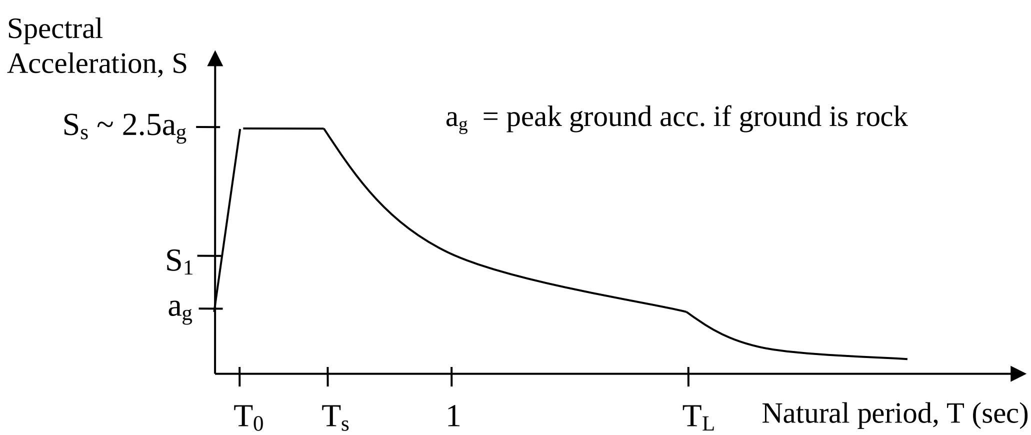


FIG. 3. Acceleration response spectrum

The base shear equation is based on the structure’s acceleration response spectrum for the case where the structure is founded on rock as shown in Fig. 3, which indicates the meaning of some of its main terms. Typically the structure is not founded on rock but on soil that may be of one of a set of possible classes. The bases shear equation caters for the modification of the peak vibration due to the presence of soil, by the use of modification factors.

The base shear equation is therefore the equation of the response spectrum for a structure that can be reasonably modeled as a single-degree-of-freedom oscillator and therefore corresponds to the first mode of vibration if the structure is modeled as a multi-degree-of-freedom structure. It is important to remember that the base shear equation is actually from structural dynamics whose understanding is therefore critical for a thorough understanding of seismic forces.

The IBC 2009 refers its users to the ASCE 7-05 "Minimum Design Loads for Buildings and Other Structures" which specifies the design base shear by the formula:

$$V = C_S W \tag{1}$$

$$C_s = S_{DS}/(R/I) \tag{2}$$

C_s is the *seismic response coefficient*; S_{DS} corresponds to the design spectral acceleration in the short period range (0.2 sec); R is the ductility factor, I is the occupancy importance factor, and W is the weight of the structure. Equation (2) represents the plateau region of Fig. 3.

There are limits on the values of C_s calculated from (2). It need not exceed:

$$C_s = S_{D1}/(T(R/I)) \tag{3}$$

for $T \leq T_L$

where T is the structure's natural period, and T_L is the long-period transition period.

C_S calculated from (2) also need not exceed:

$$C_s = S_{D1}T_L/(T^2\,(R/I)) \tag{3b}$$

for $T > T_L$

$$C_s \text{ must be} \geq 0.01 \tag{4}$$

Additionally, for structures with an S_1 greater than or equal to 0.6g,

$$C_s \text{ must be} \geq 0.5\, S_1/(R/I) \tag{5}$$

It should be noted that ASCE 7-05 allows the consideration of soil-structure-interaction (SSI) which has the effect of reducing the value obtained from equation (1).

a. THE DESIGN SPECTRAL ACCELERATIONS: S_{DS}, S_{D1}

The design spectral accelerations are the values used for calculation of the design seismic force. It should be mentioned here that this level of the seismic force corresponds to that associated with a moderate earthquake and for which the intended behaviour of the structure is such that non-structural elements do not fall on occupants, and structural elements may be repairable. This is called the life safety (LS) limit state. The collapse prevention (CP) limit state corresponds to a more intense earthquake than that associated with the LS limit state. The intended behaviour of the structure under such a severe earthquake is that the structure does not collapse, though non-structural elements will likely have fallen, and the structural elements will likely not be repairable. The code assumes that the level of seismic force corresponding to this severe earthquake, called the maximum considered earthquake (MCE) below, is about 1.5 times that associated with the LS limit state. <u>This 1.5 collapse margin has always been, and remains, inherent in the U.S codes.</u>

The design spectral accelerations also consider the amplification of the vibration of the rock (where the earthquake originates) due to the soil overlying the rock.

$$S_{DS} = (2/3)\, S_{MS} \qquad (6)$$
$$S_{D1} = (2/3)\, S_{M1} \qquad (7)$$

$$S_{MS} = F_a\, S_S \qquad (8)$$
$$S_{M1} = F_v\, S_1 \qquad (9)$$

Prior to the use of spectral acceleration to define the base shear, the ground acceleration was used. However, it was acknowledged that ground acceleration does not correlate with structural damage as well as the spectral acceleration, so the spectral acceleration is now used rather than the ground acceleration.

The "M" in equations (6) to (9) represent the fact that the soil modifies the rock vibration so the F_a and F_v are amplification factors due to the soil. The S_S and S_1, discussed in the Introduction, are the spectral accelerations of the structure on rock and shown in Fig. 3.

1) SPECTRAL ACCELERATION ON ROCK: S_S, S_1

The S_S and S_1 values are determined by reference to seismic hazard maps for the region where the structure is located. Maps for the Caribbean are derived by the University of the West Indies Seismic Research Center and freely available on their website.

These values are the accelerations expected on the structure with a 2% chance of being exceeded in any 50 year exposure time or service life, when the structure is idealized as a single mass and founded on rock. The earthquake causing the ground motion is called the *maximum considered earthquake* (MCE) and it has a return period of 2475 years. It is believed that an earthquake with this return period should be used to define the magnitude of seismic design actions likely to cause collapse of the structure. Hence the S_S and S_1 correspond to the CP limit state. However, the design seismic force corresponds to the LS limit state, therefore the CP force must be divided by 1.5 to obtain the design level force. This is the reason for the 2/3 factor in (6) and (7) (i.e. 2/3 = 1/1.5),

However this does not explain why is it necessary to use MCE accelerations, which corresponds to CP, if the design force is at the LS. It should be easier to simply use the accelerations corresponding to LS, which are for the 10%/50 years or 475-year return period. The reason for this is that the earth scientists discovered that in regions of the U.S outside of California, the ratio of CP to LS spectral accelerations is actually much higher than 1.5, being about 3.5 in New York. Therefore if the acceleration corresponding to LS is used for the design force, this force will be less than the value required for the ratio of CP to LS to be 1.5, which is what the code requires. For example, if the 10%/50 S_S acceleration at location X is 0.3g and the 2%/50 S_S acceleration is 0.9g, if it is desired that the CP to LS ratio be 1.5, then the acceleration to be used for design, and which always corresponds to the LS level, is 0.9/1.5 = 0.6g. Without using the CP level acceleration, the design would be to an acceleration of 0.3g clearly much less than the 0.6g.

The long-period transition period values, T_L, are also determined from maps for the region in question. They are not yet available for the Caribbean but should be of little consequence since the value is typically in the range of 4 to 16 seconds hence only relevant to very tall structures (i.e. > 40 stories) which are rare in the Caribbean.

2) THE SITE CLASS (A TO F) AND SITE COEFFICIENTS (F_a, F_v):

The calculation of S_{DS} and S_{D1} requires the values of the site coefficients F_a and F_v which depend on the site class. The subscript "a" refers to the period range of the acceleration response spectrum where the average acceleration is constant. Likewise, the subscript "v" refers to the period range of the velocity response spectrum where the average velocity is constant. As discussed earlier, in addition to the resonance effect of the structure due to the rock vibration, the structure is prone to additional resonance effects from the soil overlying the rock. The soil resonance magnification factors are the F_a and F_v.

The site class depends on the engineering properties of the soil and are defined as:

Average Soil Properties for Top 100 Feet (30 480 mm) of Soil Profile

Site Class	**Soil Profile Name/Generic Description**	**Shear Wave Velocity, feet/second (m/s)**	**Standard Penetration Test, (blows/foot)**	**Undrained Shear Strength, psf (kPa)**
A	Hard Rock	> 5,000 (1,500)		
B	Rock	2,500 to 5,000 (760 to 1,500)		
C	Very Dense Soil and Soft rock	1,200 to 2,500 (360 to 760)	> 50	>2,000 (100)
D	Stiff Soil Profile	600 to 1,200 (180 to 360)	15 to 50	1,000 to 2,000 (50 to 100)
E	Soft Soil Profile	< 600 (180)	< 15	< 1,000 (50)
F	Soil Requiring Site-specific Evaluation.			

TABLE OF SITE CLASS

The determination of the site class is usually done by a geotechnical engineer.

Knowing the site class, the F_a and F_v values are determined from the following tables.

Site	**Mapped Maximum Considered Earthquake Spectral Response Acceleration at Short Periods**				
Class	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*a*	*a*	*a*	*a*	*a*

a = Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

VALUES OF F_a AS A FUNCTION OF SITE CLASS AND MAPPED SHORT-PERIOD MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL ACCELERATION

Site	**Mapped Maximum Considered Earthquake Spectral Response Acceleration 1-Sec Period**				
Class	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*a*	*a*	*a*	*a*	*a*

a = Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

VALUES OF F_v AS A FUNCTION OF SITE CLASS AND MAPPED 1 SEC PERIOD MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL ACCELERATION

b. THE STRUCTURE NATURAL PERIOD, T

For building structures, the approximate natural period, T_a can be determined from the following formula:

$$T_a = C_t h_n^x \tag{10}$$

where C_t is as below, and h_n is the building height in ft.

C_t and x:-

- 0.028 for steel moment frames when they take 100% of the seismic load and are not enclosed or adjoined by more rigid components that will prevent the frame from deflecting when subjected to seismic forces; x = 0.80
- 0.016 for RC moment frames when they take 100% of the seismic load and are not enclosed or adjoined by more rigid components that will prevent the frame from deflecting when subjected to seismic forces; x = 0.90
- 0.030 for eccentric braced steel frames; x = 0.75
- 0.020 for all other structural systems; x = 0.75

For non-building structures, and for a less conservative estimate of T than is given by (10), analytical methods can be used. Rayleigh Method is one such method that is amenable to hand calculation. Many computer programs are able to calculate the natural period of a structure by an eigenvalue analysis.

If any of these analytical methods are used, the calculated value is not allowed to exceed C_uT_a where C_u depends on S_{D1} and is given by:

S_{D1}	C_u
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤ 0.1	1.7

c. THE DUCTILITY FACTOR, R

The value for the ductility factor, R, discussed in the Introduction, is determined by reference to a table which also gives the corresponding C_d and Ω_0 values. In addition, the table sets the limits for use of a particular structural system such as its height. Each structural system in common use has its associated R, C_d, and Ω_0 values stated in the ASCE 7-05. For any structural system not catered for in the table, these values must be determined by appropriate testing of the system and this includes at least, the application of reversed cyclic loads to the test specimens.

The ability to use a particular structural system at a particular location depends on its perceived level of risk in terms of the economic and social consequences of failure of the structure. This is expressed as the *Seismic Design Category* (SDC) and depends on its *Occupancy Category*, and its short and long period spectral accelerations. A system permitted for one SDC can be used for the less severe SDCs (e.g SDC C can be used for SCD B and A).

1) OCCUPANCY CATEGORY (I to IV):

The following are examples of building facilities typical for each occupancy category:

IV –Essential facilities: Hospitals and other health care facilities having emergency or emergency treatment facilities; fire, ambulance, police stations; emergency shelters; emergency preparedness centers; emergency response centers; emergency operations centers; ancillary structures that support occupancy IV structures.

III – Structures where more than 300 people congregate; daycare facilities with more than 150 persons; elementary or secondary schools with a capacity with more than 250 persons; universities and colleges with a capacity for more than 500 persons; jails and detention facilities; power generating stations; water treatment facilities; telecommunication facilities; sewer treatment facilities, not included under IV.

II - All other buildings or structures not under I, III, and IV.

I - Agricultural, minor and temporary facilities.

2) THE SESIMIC DESIGN CATEGORY (A TO F)

The SDC depends on both the spectral accelerations, and the occupancy category and is determined by reference to the following table. The higher SDC governs except if all the following conditions are met, in which case the SDC is determined by the first table only:

- $S_1 < 0.75$
- $T_a < 0.8\ S_{D1}/S_{DS}$ in each of the two orthogonal directions
- The period used to calculate drift is $< S_{D1}/S_{DS}$
- C_s is determined from equation (3)
- All diaphragms are rigid or if flexible, the distance between vertical elements of the lateral load resisting systems < 40 ft.

If $S_1 > 0.75$, an occupancy category I, II, or III structure shall be considered as SDC E, and for occupancy category IV, SDC F.

A diaphragm is a floor or roof that accepts the seismic force and transfers it by in-plane action to its supporting frames and/or walls. A rigid diaphragm is one whose span-to-depth ratio in the direction of the seismic force is less than 3 or, a rigid diaphragm is a diaphragm that when under the seismic force, its maximum in-plane deflection is less than twice the average drift of the adjoining walls and/or frames if their load is in proportion to tributary area.

	Occupancy Category		
Value of S_{DS}	**I or II**	**III**	**IV**
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

SEISMIC DESIGN CATEGORY BASED ON SHORT PERIOD RESPONSE ACCELERATIONS

	Occupancy Category		
Value of S_{D1}	**I or II**	**III**	**IV**
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

SEISMIC DESIGN CATEGORY BASED ON 1SECOND PERIOD RESPONSE ACCELERATIONS

The following is an excerpt from the IBC 2009 of R and C_d values for certain systems. Each of these systems must be detailed in a certain manner, or the values will not be achieved, which will lead to unintended and probably unfavorable response under the earthquake. Also, each system has limitations on its use, such as maximum height, that depends on the SDC. Refer to ASCE 7-05 for these details.

Basic Seismic-Force-Resisting System	**Response Modification Coefficient, R**	**Deflection Amplification Factor, C_d**
Bearing Wall Systems (System takes all gravity and lateral loads)		
Special RC shear walls	5	5
Ordinary RC shear walls	4	4
Special reinforced masonry shear walls	5	3.5
Building Frame Systems (Gravity load mainly taken by separate frame system)		
Special steel concentrically braced frames	6	5
Special RC shear walls	6	5
Moment Resisting		

Frame Systems		
Special RC moment frames	8	5.5
Special steel moment frames	8	5.5
Ordinary steel moment frames	3.5	3
Dual Systems with Intermediate Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces		
Special RC shear walls	7	5.5
Ordinary RC shear walls	6	5

R AND C_d FACTORS FOR BASIC SEISMIC-FORCE-RESISTING SYSTEMS

Note that in the above table, the term "ordinary" means ordinary ductility capacity. Such systems must still incorporate certain seismic provisions. Likewise, the term "intermediate" means an "intermediate" level of ductility capacity, as opposed to "special" which means a system that incorporates seismic provisions resulting in maximum ductility capacity.

d. THE IMPORTANCE FACTOR, I

The intention here is to represent the fact that the consequence of damage or collapse of certain structures is more significant for one type of occupancy versus another. The I factor is determined from the table below.

Occupancy Category	**I**
I or II	1.0
III	1.25
IV	1.50

e. THE STRUCTURE WEIGHT, W

The seismic dead load consists of the total weight of the structure, plus partitions and permanent equipment. It also includes 25% of floor live load in areas used for storage.

SUMMARY:

Given the aforesaid, the following is a summary of steps to be taken to calculate the base shear, V of the structure.

1. Determine the Occupancy Category (I to IV) from the table.
2. Determine the I factor value from the table.
3. Determine the S_S , S_1 and T_L values from the maps.
4. Determine the site class (A to F).

5. Determine the F_a and F_v values from the table.
6. Calculate the S_{DS} and S_{D1} from equns (6) to (9).
7. Calculate T_a from eq (10), or T and T_a .
8. Determine the SDC (A to F) from the tables.
9. Determine the R (and C_d and Ω_0) factor from the table, or from appropriate structural testing.
10. Calculate the structure weight, W
11. Substitute in equs (2) and (1) for C_s and V, respectively, and check the limits on V by equs (3) to (5).

IBC 2009 CONDITIONS OF APPLICABILITY OF THE BASE SHEAR EQUATION BY THE DETAILED PROCEDURE:

The base shear equation and allied calculations, collectively called the equivalent lateral force (ELF) procedure can only be used under certain conditions. The ELF procedure can be used for:

1. All regular or irregular buildings in SDC B or C except: occupancy category I and II buildings of light-framed construction more than 3 stories in height; occupancy category I and II buildings of any other type of construction more than 2 stories in height.
2. All regular structures in SDC D, E, or F with $T \leq 3.5\ S_{D1}/S_{DS}$.
3. All irregular structures in SDC D, E, or F but with only plan irregularity 2, 3, 4, or 5, or vertical irregularity 4, 5a, or 5b, and with $T \leq 3.5\ S_{D1}/S_{DS}$.
4. Occupancy category I and II regular or irregular buildings of light-framed construction 3 or less stories in height.
5. Occupancy category I and II regular or irregular buildings of any type of construction, other than light-framed, 2 or less stories in height.

These conditions occur when modes of vibration higher than the first translational mode contribute significantly to the total vibration.

If these conditions are not met, dynamic analysis is required (except for the case where the simplified base shear formula is applicable). In general, 3D analysis is only required for non-orthogonal buildings, or buildings that are significantly excited torsionally.

4.0 VERTICAL DISTRIBUTION OF V: F_x

After the base shear V is calculated, the vertical distribution of V is then determined by calculating the seismic force at each floor, x, termed F_x. V is the resultant of the F_xs.

$$F_x = C_{vx}\ V \tag{11}$$

$$C_{vx} = w_x\ h_x^{\,k} / \left(\textstyle\sum_i^n w_i\ h_i^{\,k}\right) \tag{12}$$

w_i, w_x = the portion of the dead load at or assigned to level i or x.
h_i, h_x= height above the base to level i or x
n = the number of floors

k = an exponent related to the building period as follows:
For buildings with a period of 0.5 seconds or less, k=1.0. If the period is 2.5 seconds or more, k=2.0. For buildings with a period between 0.5 and 2.5 seconds, it may be taken as 2.0 or determined by linear interpolation between 1.0 and 2.0.

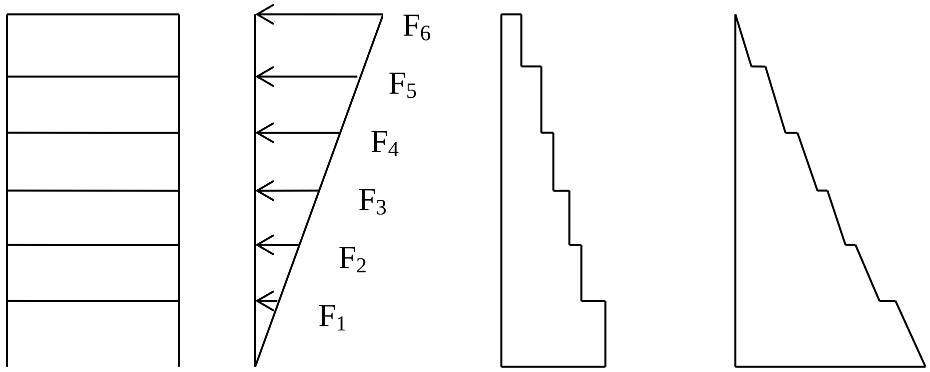
For k=1.0 the distribution is a straight line. This is reasonable for short buildings with a regular distribution of mass and stiffness. Hence, k=1.0 for buildings with a period of 0.5 seconds or less.

For k=2.0 the distribution is a parabola with the vertex at the base. This is reasonable for tall regular buildings where the participation of higher modes is significant. Hence, k=2.0 for buildings with a period of 2.5 seconds or more.

F_x acts on the entire floor at x, and through its center of mass. It should be mentioned that after distributing V vertically to each floor, the F_xs must be distributed horizontally as well to each 2D frame or wall that supports floor x in the direction of V. The seismic force exerted on each such frame or wall is in proportion to its relative stiffness if the floor is rigid. If the floor is flexible, the seismic force is in proportion to the tributary area of the frame or wall. For preliminary design, the horizontal distribution can be done by dividing the F_x for a floor, by the number of frames minus 1, in the direction of F_x, then adding 25% to the result.

5.0 STOREY SHEAR AND OVERTURNING MOMENT: V_x, M_x

F_x is the external seismic force on the structure. But the storey shear,V_x, and the overturning moment, M_x, are the resultant shear forces and moments within the structure if a horizontal section is cut at x. Since the structure under the seismic forces acts basically like a vertical cantilever, V_x and M_x decrease with height unlike the seismic forces, which increase with height. For example,



F_x V_x M_x

$V = \Sigma_1^{\,6} F_x$

The value of calculating the V_x and M_x, is that it can be used to calculate the seismic forces on the individual structural elements by hand calculation methods, as well as a structure's redundancy level (next section).

The storey shear, $V_x = \sum_{i=x}^{n} F_i$ (13)

The overturning moment:

$$M_x = \sum_{i=x+1}^{n} F_i\,(h_i - h_x) \quad ; M_n = 0 \qquad (14)$$

where,

$h_{x \text{ or } i}$ = height from the base at level x or i; n = number of floors.

6.0 SEISMIC EFFECTS IN THE STRUCTURAL MEMBERS: E

When the external horizontal seismic forces are determined as above, the subsequent structural analysis gives the stress resultants within the members (i.e. the P, M and V design actions). The set of all these internal forces, which are the <u>effects</u> of the horizontal seismic loads, is referred to as Q_E. Similarly, D is the set of all the internal forces arising as effects due to the dead load on the structure.

The effect of the vertical seismic loads is given by $0.2S_{DS}D$.

E is the total effect of both the horizontal and the vertical seismic forces, and is given by,

$$E = \rho Q_E + 0.2S_{DS}D \quad (15)$$

$$E = \rho Q_E - 0.2S_{DS}D \quad (16)$$

In the ASCE 7-05, ρ is a penalty factor for less redundant structures and is discussed below.

Equ (15) considers the case where the effect of D is additive to Q_E, and (16) when it counteracts Q_E, hence the minus sign.

For certain vulnerable structural members (e.g. those supporting discontinuous frames) The maximum or *amplified seismic load effect* is used:

$$E_m = \Omega_0 Q_E + 0.2S_{DS}D \quad (17)$$

$$E_m = \Omega_0 Q_E - 0.2S_{DS}D \quad (18)$$

a. THE REDUNDANCY FACTOR, ρ

As discussed in the Introduction, the behavior of the regular ductile moment-resisting space frame structural system is the basic reference against which the other structural systems are compared. Typical general arrangements of the structural elements of this system have multiple loads paths (i.e. redundancy) along which stresses can be transferred to other regions of the structure when yielding occurs. In the earlier U.S seismic codes, the total number of structural systems catered for in the table of R values, was much smaller, but practical interest in the other systems lead to their inclusion. However, structures comprised of these systems tend to have less redundancy compared with typical space moment frames. The effect of this is believed to be more significant for the structures more prone to seismic damage. To cater for this, a scaling factor, called the redundancy factor, ρ is applied to Q_E. The value of ρ therefore depends on the structural system, the general arrangement of the structural elements of that system, and the SDC. The following covers the most basic considerations.

ρ = 1.0 for seismic design categories B and C. For structures in SDC D, E, or F:

ρ = 1.3 but ρ = 1.0 if either of the following conditions is met:-

- Each story of a building of moment resisting frames in each direction, that is resisting more than 35% V in the direction of interest, would not lose more than 33% of the story strength if at both ends of any beam the moment resistance is lost, and by such loss the resulting system does not develop plan irregularity type 1b. Refer to the ASCE 7-05 for the conditions for other systems.
- The structure is regular in plan at all levels and has at least 2 bays of seismic force resisting perimeter framing on each side of the structure, in each orthogonal direction, resisting more than 35%V. If the perimeter grid lines has shear walls, the number of required bays with shear walls equals the length of the wall divided by the story height.

7.0 SEISMIC LOAD IN THE PRESENCE OF OTHER LOADS – LOAD COMBINATIONS

The effects of the seismic loads do not occur in isolation, but always in the presence of the effects of other forces such as the dead load, D, and the live load, L. Therefore, for design of the members, these effects must be combined. The following load combinations are used, if the element is to be designed using ultimate strength principles.

Basic seismic load combinations (for Caribbean usage):

$$1.2D + f_1 L \pm 1.0E \qquad (19)$$
$$0.9D \pm 1.0E \qquad (20)$$

where,
f_1 = 1.0 for floors in places of public assembly, or for live loads greater than 4.79 kN/m^2, or for parking garage live load. f_1 = 0.5 for all other live loads.

Equ (19) is used when the gravity loads are detrimental to the structure, and (20) when the gravity loads are beneficial (e.g. prevention of overturning or toppling of the structure).

The minus sign in front of E in (19) and (20) refers to the seismic effects when the horizontal seismic load is applied in the opposite direction. Therefore for complete structural analysis of the structure in consideration of earthquakes, there are really four seismic load combinations to be applied in each plan direction of the structure.

8.0 EXAMPLE CALCULATION FOR V, F_x, V_x, and M_x

Determine the design seismic forces for a six story special ductile moment resisting reinforced concrete office building located in Trinidad on dense soil and at a site for which $S_s = 1.3g$ and $S_1 = 0.3g$. The story heights are all 3.0 m and each floor weighs 4548 kN, except the roof which weighs 4215 kN.

Base Shear:

$V = C_S W$
Occupancy category = II
$I = 1.0$
T_L is irrelevant as the structure's height is much less than required for this portion of the acceleration response spectrum to be activated.
Site class = D
$F_a = 1.0$
$F_v = 1.8$
$S_{MS} = F_a S_s = 1.0\ (1.3g) = 1.3g$
$S_{M1} = F_v S_1 = 1.8\ (0.3g) = 0.54g$
$S_{DS} = 2/3\ S_{MS} = 2/3(1.3g) = 0.87g$
$S_{D1} = 2/3\ S_{M1} = 2/3\ (0.54g) = 0.36\ g$
$T_a = 0.016 \times (6 \times 3.0 \times 3.28)^{0.9} = 0.63$ sec
$0.8\ S_{D1}/S_{DS} = 0.8 \times 0.36/0.87 = 0.33 < 0.63$ sec. Hence use the second table only for determining the SDC.
$0.2g \leq S_{D1}$ hence for OC II, Seismic Design Category = D
R = 8 (special moment frame)

$C_s = S_{D1} I/(RT) = 0.36 \times 1/(8 \times 0.62828) = 0.071624$
$S_{DS} I/R = 0.87 \times 1/8 = 0.11 > C_s$
Hence $C_s = 0.072 > 0.01$:OK
$W = 5 \times 4548 + 4215 = 26955$ kN
$V = 0.071624 \times 26955 = 1930.6$ kN

Vertical Distribution (F_x), V_x and M_x:
$F_x = C_{vx} V$
$C_{vx} = w_x h_x^k / (\sum_i^n w_i h_i^k)$
T > 0.5 therefore by interpolating between k = 2 @ 2.5s, and 1.0 at 0.5s, k = 1.065 @ 0.63s

$$V_x = \sum_{i=x}^{n} F_i$$

$$M_x = \sum_{i=x+1}^{n} F_i (h_i - h_x) \quad ; M_n = 0$$

The following table shows the calculations for V_x and M_x which are readily performed using a spreadsheet.

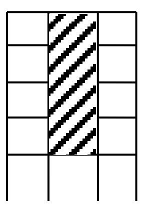
Level, i	w,i	h,i	(h,i)^k	w,i (h,i)^k	**Fx**	Vx	Mx
	kN	m			**kN**	kN	kNm
6	4215	18	21.72	91550.94	**536.31**	536.31	0
5	4548	15	17.89	81350.02	**476.55**	1012.85	1608.92
4	4548	12	14.10	64142.88	**375.75**	1388.60	4647.48
3	4548	9	10.38	47215.95	**276.59**	1665.19	8813.29
2	4548	6	6.74	30658.54	**179.60**	1844.79	13808.87
1	4548	3	3.22	14653.95	**85.84**	1930.63	19343.25
0		0				1930.63	25135.15
	26955			329572.28	**1930.63**		

IRREGULAR STRUCTURES

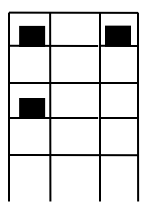
The following is the ASCE 7-05 classification of irregularities. Refer to ASCE 7-05 for for the specific quantitative criteria that defines each type of irregularity. The ASCE 7-05 also quantifies the degree of severity of the irregularity in that depending on the type of irregularity, the structure is allowed to be used only for certain SDCs, and only under certain conditions. Furthermore, if the structure is irregular there are additional considerations that must be made for the determination of the design actions of its structural elements. Refer to ASCE 7-05 for this information.

<u>Vertical Irregularities:</u>

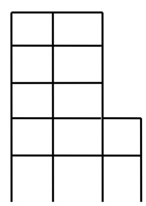
Type 1a,b
Soft Storey



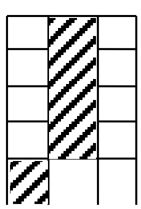
Type 2
Mass Irregularity



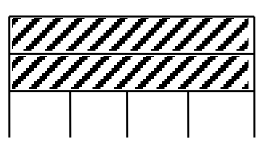
Type 3
Geometry Irregularity



Type 4
In-plane offset

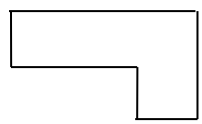


Type 5
Weak Storey

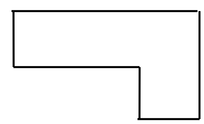


Plan Irregularities:

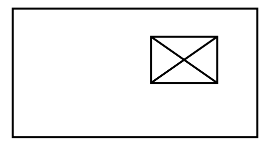
Type 1a,b
Torsional
Irregularity



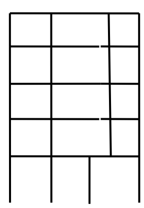
Type 2
Re-entrant corners



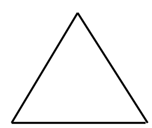
Type 3
Diaphragm Discontinuity



Type 4
Out-of-plane
offset



Type 5
Non-Parallel Systems



BASE SHEAR, V - THE SIMPLIFIED PROCEDURE

The simplified procedure eliminates the need to determine certain parameters such as the drift and irregularity but the simplified procedure only applies to structures that are: less than 3 stories in height; OC I or II; of SDC A to D, and that are not drift-controlled (e.g. bearing wall and building frame systems). Otherwise, the detailed procedure must be used., as described in this presentation

Refer to ASCE 7-05 for further information on the simplified procedure.