B2.1 Outline of Analytical Procedures I

by Tony Gibbs, BSc, DCT(Leeds), FICE, FIStructE, FASCE, FConsE, FRSA
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1 Qualitative Analysis

Increasingly computers are taking over the calculation tasks in quantitative structural analysis. In the case of younger engineers who grew up with computers, there is increasing competence in the use of computers and a corresponding regression in the understanding of structural behaviour. This can be dangerous. This widening gap needs to be bridged. The bridge is the greater attention to (and the teaching of) qualitative analysis. This would bring back the "feel" of structures to engineers and would facilitate their "seeing" the structural behaviour.

Sophisticated rigorous analysis based on false application of theory has resulted in erroneous conclusions and failure of structure.

Failure to develop a better understanding of structural behaviour would lead to poorer designs and more errors, some of which may lead to catastrophes. A better understanding of structural behaviour would be an important safeguard against the misuse or misunderstanding of the application of computers in structural engineering.

Although the problem identified here is more prevalent among younger engineers, even engineers of many years experience often have difficulty in applying their understanding of structural behaviour when confronted with the task of modelling a structure for computer analysis.

Modern steel structures are almost always analysed by computer. Indeed steel design[1] is now being done by computer. Consequently, there is a need for better understanding of structural behaviour which would lead to better modelling of structures as input for the computing machine.

In the words of Dr David Brohn[2]:

> "....a proficiency in numerical analysis does not appear to confer an equivalent understanding of structural behaviour. This overall understanding of materials and the structure under load becomes of primary importance in the first stages of

[1] "Design" is taken in the sense of determining the preliminary sizes of the members.

[2] David Brohn's career included research, teaching, consulting, writing and software development.

conceptual design, in the creation of the analytical model and in the checking of the computer output."

The problem is compounded by the pressures of fee competition and impatient clients which both lead to a reduction (or elimination) of the structured training of graduates in engineering.

The approach to be aimed at is:

- apply a qualitative approach to the approximate analysis of the structure
- create the structural model for computer analysis
- check the computer input in an orderly and thorough way
- compare the computer output with the crude results from the approximate analysis

2 Typical Problems and Errors

2.1 General Problems not Limited to Natural Hazards

The fundamental issue encountered relates to an understanding of how structures deform under load. This is often combined with a difficulty in determining the directions of bending moments and the signs (+ or -) of internal forces.

Lack of experience hampers the ability to determine the orders of magnitude of forces and moments and the ability to estimate appropriate preliminary sizes for members. This leads directly to blind belief in computer results.

All engineering is based on simplifying assumptions. These are usually stated at the start of a course (*eg* "plane sections remain plane" and "the geometry is unchanged by the loading"). After a period of time the assumptions are forgotten. When a problem is encountered where the assumption is seriously invalid, no change is made to the analytical procedure.

Another type of assumption which often leads to distress is where the analytical assumption is not mirrored in the detailing. The most common example of this is in modelling end conditions as pinned (or fixed) in the analysis without being able to (or attempting to) achieve this in the detailing and construction.

Visualising structures in three dimensions can also pose problems for many persons. (Most structural analyses is of two-dimensional systems.)

2.2 Lessons from The IStructE

The Institution of Structural Engineers' Part 3 examination for Chartered status is regarded as one of the best international, code-neutral tests of competence for structural engineers. A review of the examiners comments after the marking of papers each year indicates several common mistakes. Some of these are reproduced below from an IStructE publication[3]:

- The reinforced concrete frame (see figure l), was reinforced incorrectly. Whilst the calculations, the member sizes, area of reinforcement, *etc*, were 'accurate' the detailing was not. This not uncommon example demonstrates a lack of ability to 'see' structural deformation.

- The 8m deep reinforced concrete pit (see figure 2) subject to earth pressure was not designed as a series of horizontal box sections but as a series of vertical, free cantilevers.

- Frequently trusses and portal frames are accurately designed in-plane but lack bracing in the transverse plane and thus can be unstable. Typical was the collapse of trussed timber rafters at the Sports Hall at Rock Ferry Comprehensive School, Birkenhead. Temporary bracing during erection of steel and precast concrete frames (before stiffening by cladding, *etc*, is added) is occasionally omitted and has resulted in construction collapse.

- A crane gantry was required to carry extra load and the gantry beams were checked and found adequate. The loading was increased and the beam connection and supporting columns (which had not been checked) failed under the increased load.

- A precast three-pinned arch was designed to accommodate differential settlement due to mining subsidence, (see figure 3 (i)). A mezzanine floor was added to some arches and the advised pinned joints, fig 3 (ii), were substituted by fixed joints, fig 3 (iii). The mezzanine frames in fig 3 (iii) suffered extensive and expensive failure due to differential settlement but no damage was caused to the three-pinned arches.

- The structural elements of the octagonal structure, shown in figure 4, were adequately designed by individual specialists but there was no overall check on stability and connection. The individual elements were structurally sound but the roof suffered dangerous vertical deflections due to lack of adequate interconnecting joints.

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[3] Qualitative Analysis of Structures - A Report by a Sub-committee of the Education and Examinations Committee

- Buckling of compression flanges of beam elements due to inadequate consideration of lateral stiffness.

- The inability to appreciate that sudden release of pent-up strain energy in prestressed members can lead to catastrophic collapse.

2.3 Modelling for Computer Analysis

Common mistakes made in describing the structure to the computer are:

- incorrect or unrealistic assumptions on the degree of fixity of foundations;

- incorrect support conditions for secondary beams in steel-framed structures;

- neglecting to include diaphragms or omitting them to avoid larger computer models and longer computer runs;

- omitting non-structural stiff elements such as in-fill block walls;

- allocating incorrect l/r or k factors for columns.

The following session will be devoted to the use of a popular general purpose powerful program for structural analysis.

3 Methods of Seismic Analysis for Structures

Analysis is required to determine the moments, shears, direct forces and torsions in the members of a structure. The analysis will also provide deflexions. In many cases of seismic analysis and a few cases of wind analysis for buildings the dynamic response is critically important. Since the problems of analysis are typically greater for earthquakes than for hurricanes, this section will concentrate on the former.

Seismic analysis can conveniently be sub-divided into:

- equivalent static force analysis

- dynamic analysis

3.1 Equivalent Static Force Analysis

Equivalent static force analyses are approximations (often gross approximations) of reality which are used for the vast majority of buildings because of the great difficulties associated with realistic (or any credible) dynamic analysis. All loading and design standards and codes for buildings permit equivalent static force analysis for a greater or lesser range of structures. They all start from the simple basis of:

force = mass multiplied by acceleration
which for earthquakes is:

(horizontal base shear) = (fixed mass of the building) x (seismic horizontal acceleration)
or
$$V = ma$$

What happens next is a series of refinements so that results will approach those obtained from realistic dynamic analyses.

The first and common refinement is to provide rules for distributing the total base shear vertically over the entire building height. This is usually a triangular distribution with or without an additional point load at the top of the building.

A refinement introduced by ATC-3[4] eliminated the point load but curved the hypotenuse of the triangle.

The equivalent static force analysis then takes these distributed forces and determines the resulting moments, shears, *etc* by any conventional means.

Traditionally a lower load factor (or, in the past, factor of safety) is applied to earthquake forces than to gravity live loads. This is so for wind forces also. The problem is that the simplifying standards and codes have already reduced the "real" earthquake accelerations considerably in the interests of economy. So that even the factored earthquake loads are usually much less than those expected in the "design event".

If the wind forces turn out to be more than the "code" earthquake forces, it is quite common for the designer to pay no further attention to the earthquake hazard as he proceeds with his detailing. This is a mistake and, in some cases, a grave mistake. Even when design wind forces exceed "code" earthquake forces, the detailing may be controlled by the earthquake requirements for ductility as well as the need to accommodate possible greater deflexions in the subject earthquake.

3.2 Dynamic Analysis

Various methods of dynamic analysis have been developed for use in building structures. They include:

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[4] Applied Technology Council - Tentative Provisions for the Development of Seismic Regulations for Buildings.

- direct integration of the equations of motion by step-by-step procedures;
- normal mode analysis;
- response spectrum techniques;
- power spectrum;
- time history.

This issue will be dealt with further in a later session of this course.

3.3 "Partial" Dynamic Analysis

An important step in carrying out equivalent static force analysis in accordance with modern standards is the determination of the fundamental period of the structure. Such standards always provide relatively simple methods and formulae for computing approximate answers for the periods. On the other hand, several general purpose analysis computer programs, readily available to designers, can compute more accurately periods of structures.

Therefore a refinement of the equivalent static force analysis method is the determination of the period of the structure by dynamic analysis followed by a reversion to conventional equivalent static force analysis.

The approach:

- apply a qualitative approach to the approximate analysis of the structure

- create the structural model for computer analysis

- check the computer input in an orderly and thorough way

- compare the computer output with the crude results from the approximate analysis

Common mistakes:

- incorrect or unrealistic assumptions on the degree of fixity of foundations;
- incorrect support conditions for secondary beams in steel-framed structures;
- neglecting to include diaphragms or omitting them to avoid larger computer models and longer computer runs;
- omitting non-structural stiff elements such as in-fill block walls;
- allocating incorrect l/r or k factors for columns.

B2.2 Outline of Analytical Procedures II

An Introduction to STAAD III/STAADPro

by Anthony C T Farrell, MSc, FICE, FIStructE
November 2000

1.0 Introduction

With the increasing complexity of analytical requirements of seismic engineering it is almost mandatory for a structural engineer to utilize some form of computer program as a tool.

STAAD is one of a number of general purpose structural analysis computer programs. There are many others e.g. SAP IV, Robot, STRAP to name a few.

This class of program is suitable for all types of structure unlike special purpose programs like RAMsteel and ETABS which suit particular types of structure or LEAP which suits bridges. The general purpose programs will all have similar features in principle.

I intend to illustrate some of the features of STAAD as an introduction to the approach of general purpose structural analysis and design programs.

STAAD (which actually stands for Structural Analysis And Design) apparently started life as STAAD III approximately 20 years ago and has improved in functionality and reliability steadily. The latest version is STAAD Pro 2000 which is extremely flexible but is quite difficult to learn thoroughly. STAAD III/Pro is developed and marketed by Research Engineers Incorporated who have offices in many countries. Their Head Office is in California, USA

2.0 Description

The processing part of the program requires a series of statements resembling brief English words almost like a simple language. These statements all have their own syntax.

An illustration of a typical listing of the command structure of a STAAD input file is given in section 6.0 of this document. Words in curly brackets are a choice of mandatory words. Words in simple brackets are optional. Letters underlined are the minimum letter abbreviations for the words accepted as correct syntax by the program.

Appendix I contains a copy of a simple STAAD file output. The geometry of the structure is shown in Figure 1.

Each basic command is followed by a list of data relating to that section of input. Most of the commands are mandatory and if omitted will generate an error when the program is run. The design section is optional.

Our younger engineers prefer to use the graphical methods allowed for input but there are situations where the writing of a file is more efficient. The graphics input operates similar to a Windows wizard but is not quite as organized. At the end of graphical input the program automatically converts this information to a written file in the same format that has been outlined before analysis can be carried out.

3.0 Elements Allowed

There are only two types of element allowed by the program viz. linear elements (called members) and finite elements (called elements). There are three different types of element allowed viz. three sided (three nodes), four sided (four nodes) and eight sided block elements (eight nodes). The last kind is used only for very special types of problem. The usual 3 and 4 sided elements are assumed to be thin plates but STAAD uses a hybrid formulation of element in finite element analysis which allows the program to design for out-of-plane bending moments. Thus we often design slabs of unusual geometry or even of regular geometry by using these elements. These elements would also be used to model shear walls and floor diaphragms. Many special purpose finite element programs do not have this feature allowing for out-of-plane bending.

All structures are built up using the basic member/element types.

4.0 Output

There is a truly vast flexibility allowed for output both printed and graphic so I will not attempt to go into them in this brief introduction. The user can see BMDs and deflection diagrams of part or all the analysed structure, stress diagrams in colour of finite elements, lists of forces stresses and deflections sorted in any order and for any list of load combinations specified by the user and many more things some of which I have never been able to use.

5.0 Projects & Problems

Following are some illustrations of different projects which have been analysed using STAAD (Fig. 2 to 4).

Some things that require care follow:

1) Column fixity and member releases (Fig. 5 & 6)
2) Correct allocation of masses for UBC or dynamic analysis
3) Diaphragms for 3-D structures
4) Use of effective member lengths for design of steel members. (Fig. 7)

6.0 Typical Layout of a STAAD File

STAAD{SPACE, PLANE, TRUSS, FLOOR} (any title a_1)

UNITS{length, force}

JOINT COORDINATES (CYLINDRICAL, (REVERSE)) (NOCHECK) band-spec

MEMBER INCIDENCES

ELEMENT INCIDENCES

ELEMENT INCIDENCES SOLID

MEMBER PROPERTIES {BRITISH, AMERICAN, CANADIAN etc}

Member list {TABLE, PRISMATIC, TAPERED, UPTABLE i, ASSIGN}

[note: PRISMATIC can be T-Sect or Trapezoidal]

ELEMENT PROPERTY
element-list THICKNESS f_1 , (f_2, f_3, f_4)

MEMBER RELEASE
member-list {START, END} {six degrees of freedom}
Note Partial release MP also available

ELEMENT RELEASE {J_1, J_2, J_3, J_4} {six degrees}
Note 1) Also Member truss cable, Tension, Compression available.
Note 2) Also Element plane stress and ignore inplane rotation available

CONSTANTS {E, POISSON, DENSITY, BETA, ALPHA} f_1{MEMBER list, ALL}

SUPPORTS
joint list {PINNED, FIXED (BUT release or spring)}

Inclined support available
Automatic spring support available for footings or mats
Master/Slave Specification

Definition of Load Systems

Moving Loads, UBC loads (1985 & 1994 & 1997) with or without accidental torsion, Wind loads, Time history loads, Joint Loads, member loads, element loads, Area load, Floor Load, prestress loads (pre & post-tension), Temperature, fixed end, support-displacement, Selfweight, Dynamic loading (response-spectrum, time history)

<u>LOA</u>D <u>GEN</u>ERATION

For moving loads, UBC, Wind,

LOADS

<u>REP</u>EAT <u>LOA</u>DS

<u>CAL</u>CULATE <u>NAT</u>URAL (FREQUENCY) – Rayleigh iteration

<u>MOD</u>AL (CALCULATION) - Full eigensolution

<u>LOA</u>D <u>COM</u>BINATION (SRSS) i a

<u>PRI</u>NT <u>PRO</u>BLEM <u>STA</u>TISTICS

<u>PER</u>FORM or <u>PDE</u>LTA(n) or <u>NON</u>LINEAR (n) <u>ANA</u>LYSIS

Note non-linear takes geometric non-linearity into account

<u>LOA</u>D <u>LIS</u>T {load-list, all}

PRINT {many alternatives} {(ALL) or LIST}

PLOT {DISPLACEMENT, BENDING, STRESS} FILE
MODE

Design

Many codes supported, steel and concrete

PARAMETERS or START CONCRETE DESIGN
follow by list of parameters
CHECK CODE or DESIGN{BEAM COL, ELEMENT}

END DESIGN

FINISH

B2.3 Introduction to Dynamic Analysis

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November 2000

1.0 Introduction

This paper is intended purely as a brief introduction to some of the basics of dynamic analysis by a practicing engineer. This therefore is a non-specialist view of some of the basics. It deals mainly with linear elastic behaviour of single degree of freedom (SDOF) systems.

2.0 Response

The response of a structure to an earthquake may refer to stress, displacement, acceleration, velocity, shear or any other parameter affected by ground motion. Response may be defined in time but it is customary to refer to the response as the peak value of the particular parameter caused by the earthquake.

In general, the effect of vibration on a structure can be calculated - if a numerical model can be arrived at - of both the vibration or "forcing function" and the structure. Methods for the forcing function are:

A – Time-based records
B – Response spectrum
C – Power spectrum

The linear structural model requires information on structural stiffness, derived from the geometric and material properties in the same manner as for static analysis as well as information on the masses and their distribution since seismic dynamic forces derive from the inertia of structural masses.

3.0 SDOF Systems

Real structures have an infinite number of DOF but are often modeled as simplified models (Figure 1) with a discreet number of degrees of freedom. This presentation does not go into the derivations but suffice it to say that there is a matrix formulation and solution so that for an n degree of freedom system there will be n equations, n mode shapes and n modal frequencies.

SDOF systems mathematics is used to evaluate the effects on a structure of each mode with the effects of the separated modes being combined to evaluate the overall effect on the structure. In general the first few nodes will dominate so that only those few need be summed. This summation is not arithmetic since the peak responses usually do not occur at the same time. One method of summing is to use the square root of the sum of the squares.

Examining SDOF systems is also useful for illustrating the distinction between natural damping and added damping, how damping affects the response of structures as well as the principles of dynamic magnification and resonance, and their influence on structural behaviour.

4.0 Idealized SDOF System

Most lecturers of structural dynamics idealize this as the SDOF mass-spring-dashpot model (Figure 2). This idealization is appropriate in many cases and as mentioned before MDOF systems may be separated into several SDOF systems with effects combined.

The representation of damping as a simple viscous dashpot is commonly used as it allows for a linear dynamic analysis. Some other types of damping models require non-linear analysis.

Equations of motion (or more accurately dynamic equilibrium) can be shown as a force-balance (Figure 3) where dynamic equilibrium is always maintained while at any point in time the inertial, damping and elastic resisting forces do not necessarily act in one direction.

For a linear system the resisting forces are proportional to the motion (Figure 4). The slope of the inertial-force vs acceleration curve is equal to the mass. Similarly slope is the damping for damping-force vs velocity and slope is equal to stiffness for elastic-force vs displacement.

Figure 5 shows the equations of motion in terms of displacement, velocity, acceleration and the constant stiffness, damping and mass. Please note that stiffness and damping in a real structure may change due to damage. Taking account of this requires non-linear analysis.

5.0 Mass, Damping, Stiffness

Mass is always assumed constant throughout the response.

In analysis an equivalent viscous damper is usually assumed because of mathematical convenience (damping force proportional to velocity). However in addition to the real or natural damping arising from the material real structures can have added damping with the use of built-in damping devices. Natural damping can change with damage.

The force-displacement relationship for a linear viscous damper is an ellipse (Figure 6). The area within the ellipse is the energy dissipated by the damper. The greater the energy dissipated, the lower is the potential for damage to the structure. This is the primary motivation for the use of added damping systems.

Figure 7 illustrates the difference between energy dissipated which is not recoverable and energy absorbed (e.g. elastic strain energy) which is fully recoverable.

It is assumed that the force-displacement relationship (stiffness) is linear elastic. However real structures, especially those designed according to current earthquake code provisions will not remain elastic in large earthquakes. Despite this, elastic analysis is most often used with the structure dissipating energy by accepting a degree of damage in yielding but avoiding collapse with proper detailing.

6.0 Effective Earthquake Force

Figures 8 and 9 illustrate the development of the effective earthquake force. It is important to note that for the earthquake problem there is no actual external force applied to the structure. The forces mandated in codes and applied as external loads are just a convenient gross approximation of structural behaviour.

7.0 Undamped Free Vibration

Figure 10 illustrates the simplest problem to solve i.e. undamped free vibration. This type of response can be invoked by imposing a static displacement and releasing the structure with zero initial velocity. The equation of motion is a second order differential equation with constant coefficients. Displacement is the primary unknown. The quantity w is the circular frequency of free vibration of the structure.

Figure 11 illustrates the relationship between circular frequency, cyclic frequency and period of vibration. The period of vibration is used in the development of earthquake code provisions.

The undamped motion will continue forever if uninhibited but in real structures damping does reduce the free vibration to zero after some number of cycles.

Figure 12 shows typical periods of vibration for simple building structures. For relatively rigid structures, such as dams and nuclear power containment domes the period of vibration may be less than 0.05 sec. Period increases with increasing mass and decreases with increases stiffness.

Period is a very important parameter in the seismic resistant design of structures.

8.0 Damped Free Vibration

Figure 13 illustrates the damped response in free vibration which is similar in form to the undamped response but with an exponential decay term that serves as a multiplier on the whole response.

Critical damping is the amount of damping that will produce no oscillation. Note that damped circular frequency for most normal structures is effectively the same as the undamped circular frequency since E (fraction of critical) is usually very small (less than 0.1). Damping ratio E is given in terms of percent critical.

Figure 14 illustrates how few cycles are required for the free vibration to be effectively damped out. The difference between natural damping and added damping should be emphasized. Natural damping is a structural material property independent of mass and stiffness and in real structures varies from 0.5% to 7.0% of critical. Added damping is also a structural property but dependent on mass and stiffness and can vary from 10% to 30% critical.

9.0 Undamped Harmonic Loading

Figure 15 illustrates simple harmonic loading as a "forcing function". Figure 16 sets up the equation of motion for undamped harmonic loading and gives the solution assuming the system is initially at rest.

Figure 17 breaks the response into the steady state response (at the frequency of loading) and the Transient Response (at the structures own natural frequency). The following should be noted:

a) The term p_o/k is the static displacement
b) The dynamic magnifier shows how the dynamic effects may increase (or decrease) the response.
c) This magnifier goes to infinity as the frequency ratio goes to 1. This is defined as resonance.

Figure 18 is an enlarged view of a total response curve with the structure almost at resonance. The response bounded by a linear increasing envelope speaks for itself.

Figures 19 and 20 show the ratio of the steady state response to the static displacement for the structure loaded at different frequencies. At low loading frequencies the ratio is 1.0 (i.e. very nearly static). At very high frequency loading the structure does not have time to respond and the displacement approaches zero. Also resonance is illustrated.

10.0 Damped Harmonic Loading

Damping is introduced in Figure 21 with Figure 22 illustrating the solution to the differential equation. The transient response (A and B coefficients) will damp out and are dropped since the exponential decay term causes a rapid decrease in the transient response.

Figure 24 shows the response of a structure at three different loading frequencies. The damped response is now limited.

Figure 25 shows the dynamic magnification for various damping ratios. Damping decreases the resonant response significantly and for viscously damped structures the resonance amplitude will be limited to R_D = 1/(2 times the damping ratio). Nevertheless this is a condition to be avoided in structural response.

11.0 General Dynamic Loading

The case of SDOF systems subject to general dynamic loading may be solved in the time domain by the famous Duharmel's integral or by time-stepping methods.

Duharmel's Integral works by dividing the general load into an infinite number of very small pulses for which the responses are summed. In the language of calculus this summation is the integral.

Time-stepping methods are typically carried out numerically, one of the most efficient of which is the piece wise exact method which is based on the solution of a damped system to a short duration linear force. This is useful in earthquake analysis as ground acceleration time histories are typically represented as piece-wise linear functions.

12.0 Methods of Solution

In general methods of solution are placed into two categories viz deterministic (or in the time domain) and probabilistic (or in the frequency domain). The time domain solutions result in specific answers as opposed to the frequency domain solutions which deal with vibrations using probabilistic concepts with the basic assumption being that the statistical nature of the vibration – mean value, root-mean-square amplitude, frequency distribution – does not change with time.

13.0 Elastic Response Spectra

Figures 28 and 29 illustrate how an elastic response spectrum is developed. Using the methods discussed earlier it is possible to obtain the response of a SDOF system at any time for a specific earthquake. For design purposes the peak absolute values during the earthquake are required and these when plotted give the response spectrum.

The pseudo-velocity and pseudo-acceleration values are related to the displacement as shown. The "pseudo" prefix means that the quantity is derived and is not precisely the same as the actual related velocity and acceleration. Within the range of frequencies normally encountered in building design the distinction between pseudo and actual is not significant.

Pseudo-acceleration, when multiplied by the mass of the structure, is a measure of the shear in the structure and building codes use pseudo-acceleration spectra in the development of predicted maximum base shear.

Actually these spectra are often drawn on tripartite paper with all three items plotted against period.

14.0 Design Response Spectra

A response spectra for one earthquake is no good for design because different earthquakes produce significantly differing spectra not to mention the influence of soil properties affecting response at a specific site.

Design spectra are smoothed and will represent an estimate of response for a particular probability of occurrence based on a large number of earthquake records.

Figure 30 illustrates the NEHRP recommended provisions. These provisions include soil effects based on work done by Bolton Seed (Figure 31).

The NEHRP provisions have been incorporated into the International Building Code 2000 (IBC 2000) about which we will all be hearing a great deal in the future.

B2.4 Introduction to IBC 2000 (NEHRP[5]) Earthquake Loads

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November 2000

1 Background

There is a possibility (probability) that the next edition of CUBiC will adopt by reference (with Caribbean application amendments) IBC 2000. The earthquake design provisions of IBC 2000 follow closely the 1997 Edition of "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures"[6].

This is the justification for including an introduction to IBC 2000 (and therefore NEHRP in this course.

2 Site Ground Motion

The definition of level of earthquake hazard in IBC 2000 is a significant departure from the standards in current use in the Caribbean. The current "zones" of CUBiC, SEAOC, SBCCI and UBC are different in a meaningful way from IBC 2000.

IBC 2000 contains the concept of the Maximum Considered Earthquake (MCE) and Spectral Response Accelerations (SRA). Maps are provided for MCE-SRA at short periods (S_s) of 0.2 second and long periods (S_l) of 1.0 second. Unfortunately the IBC 2000 maps do not include the Commonwealth Caribbean. They do include Puerto Rico and the US Virgin Islands. Here are the figures for those places:

Place	**MCE 0.2 sec SRA** (5% critical damping)	**MCE 1.0 sec SRA** (5% critical damping)
Puerto Rico	100%g	40%g
USVI	150%g	60%g

These figures are for Site Class B (rock).

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[5] National Earthquake Hazards Reduction Program

[6] Federal Emergency Management Agency publication FEMA 302 / February 1998

It is expected that the SRU-UWI will be developing Caribbean-wide maps with these values by the end of 2001.

2.1 Site Class

Soil response to seismic waves is a critical determinant of site-specific effects.

Where the soil shear wave velocity is not known the Site Class may be determined from standard penetration resistance (SPR) using the standard penetration test (SPT). The seven classes (A to F) are related to SPRs and other soils mechanics parameters in a table in IBC 2000. (OHPT[7]-1)

MCE-SRA at short periods (S_s) of 0.2 second and long periods (S_l) of 1.0 second are adjusted for Site Class and are thus designated (S_{Ms}) and (S_{Ml}).

$$S_{Ms} = F_a S_s$$

$$S_{Ml} = F_v S_l$$

where:

F_a = site coefficient (short period) (OHPT-2)
F_v = site coefficient (long period) (OHPT-3)

2.2 Design SPA

The 5% damped design spectral response accelerations are determined from the formulae:

$$S_{Ds} = (2/3)\ S_{Ms}$$

$$S_{Dl} = (2/3)\ S_{Ml}$$

[7] Overhead projector transparency

2.3 Design Response Spectrum

The spectrum (ordinate Spectral Response Acceleration S_a; abscissa Fundamental Period of the Structure in seconds T) is then developed as follows:

1 For T less than or equal to T_O:

$$S_a = 0.6\ (S_{Ds}/T_O)\ T + 0.4\ S_{Ds}$$

where:

$T_O = 0.2\ (S_{Dl}/_{Sds})$

2 For T greater than or equal to T_O and less than or equal to T_S:

$$S_a = S_{Ds}$$

where:

$T_S = S_{Dl}/S_{Ds}$

3 For T greater than T_S:

$$S_a = S_{Dl}/T$$

(OHPT-4)

2.4 Site Specific Determination

IBC 2000 also gives procedures for determining ground motion accelerations for:

- the probabilistic MCE
- deterministic limit on MCE

3 Use, Occupancy and Importance

There are three Seismic Use Groups in IBC 2000:

- III - essential facilities required for post-earthquake recovery and buildings containing substantial quantities of hazardous materials
- II - buildings whose failure would result in a substantial public danger due to occupancy or use
- I - all other buildings

4 Seismic Design Categories

The Seismic Design Categories in IBC 2000 depend on the design SRAs (S_{Ds} and S_{Dl}) and the Seismic Use Group.

The categories range from A (least severe) to D (more severe). They are presented in a matrix table (OHPT-5). In addition there are Categories E and F for situations where S_l is greater than or equal to 0.75g.

Category A would have S_{Ds} less than 0.167g and S_{Dl} less than 0.067g for all Seismic Use Groups. Category D would have S_{Ds} equal or greater than 0.50g and S_{Dl} equal or greater than 0.20g for all Seismic Use Groups and S_{Dl} between 0.20g and 0.133g for Seismic Use Group III.

The least severe category (A) can be treated in the following very simple way:

- provide a complete lateral-force-resisting system
- apply lateral force F_x at each floor level ($F_x = 0.01\ w_x$) where w_x is the portion of the gravity load (W) assigned to level x
- interconnect all parts of the structure
- anchor all concrete and masonry walls (the building is considered to be fixed at its base)

5 Irregularities and Permitted Methods of Load Analysis

Plan structural irregularities and vertical structural irregularities are extensively categorised with each category attracting different restrictions and requirements for detailing and analysis.

5.1 Simplified Analysis

Simplified analysis is restricted to structures in Seismic Use Group I of light frame construction not more than 3 storeys or of any form of construction not exceeding 2 storeys.

The simplified analysis is summarised as follows:

- Seismic base shear $V = 1.2\ (S_{Ds}/R)\ W$ where R is the Response Modification Factor obtained from a table in IBC 2000 (The more favourable the system and material the higher the R. A sample of values is given in a table at the end of this paper.)
- distribute the lateral force F_x at each floor level by the formula $F_x = 1.2\ (S_{Ds}/R)\ w_x$ where w_x is the portion of the gravity load (W) assigned to level x

- assume a design storey drift of 1% for the purposes of determining building separations and deflexions for detailing appendages

- anchor all concrete and masonry walls (the building is considered to be fixed at its base)

5.2 Equivalent Lateral Force Procedure

An equivalent lateral force procedure is permitted for Seismic Design Categories B and C. In this procedure:

$$V = C_s W$$

where:

C_s = the seismic response coefficient

The seismic response coefficient is obtained from:

$$C_s = S_{Ds}/(R/I_E)$$

where:

I_E = the Seismic Occupancy Importance Factor which varies from 1.0 to 1.5.

The seismic response coefficient need not exceed:

$$C_s = S_{Dl}/\{(R/I_E)\ T\}$$

An approximate value of T (T_a) may be obtained from:

$$T_a = C_T h_n^{0.75}$$

where:

C_T = the Building Period Coefficient which varies from 0.035 to 0.020
h_n = height in feet of the building

Alternatively:

$$T_a = 0.1\ N$$

where:

N = number of storeys
for concrete and steel moment frames not exceeding 12 storeys maximum 10-foot storey height

The vertical distribution of seismic forces is determined from:

$$F_x = C_{vx} V$$
and
$$C_{vx} = w_x h_x^k/(\Sigma w_i h_i^k)$$
where:

F_x = the lateral force at level x
C_{vx} = vertical distribution factor
V = total design lateral force or shear at the base of the building
w_x and w_i = the portions of W assigned to levels x and i
h_x and h_i = heights in feet to levels x and i
k = a distribution exponent related to the building period as follows:

- k = 1 for buildings with T less than or equal to 0.5 seconds
- k = 2 for buildings with T more than or equal to 2.5 seconds
- interpolate between k = 1 and k = 2 for buildings with T between 0.5 and 2.5

The above formulae are virtually identical to those in CUBiC.

IBC 2000 gives guidance on horizontal shear distribution depending on the rigidity of the diaphragms and torsion.

The distribution of forces to the resisting elements for overturning is consistent with the horizontal shear distribution.

The P-Delta effect receives special attention. Its treatment depends on the Seismic Design Category of the building and the Occupancy Importance Factor.

P-Delta effects on storey shears and moments, resulting member forces and moments, and storey drifts induced by these effects are not required to be considered when the stability coefficient (θ) is not greater than 0.10 as determined by:

$$\theta = P_x \Delta / (V_x h_{sx} C_d)$$
where:

P_x = the total unfactored design load at and above level x
Δ = design storey drift occurring with V_x
V_x = the seismic shear force acting between levels x and x-1
h_{sx} = the storey height below level x
C_d = the deflexion amplification factor given in a table in IBC 2000

5.3 Dynamic Analysis

Dynamic analysis is required for all buildings in Categories D, E and F. The methods of dynamic analysis permitted are:

- modal response spectra analysis
- linear time history analysis
- non-linear time history analysis

6 Other Issues

The other issues dealt with in IBC 2000 are:

- soil-structure interaction
- detailing
- structural component load effects
- architectural, electrical and mechanical components

Basic Seismic Force Resisting System	Response Modification Coefficient R
Bearing Wall Systems	
ordinary steel concentrically braced frames	4
ordinary reinforced concrete shear walls	4
intermediate reinforced masonry shear walls	3.5
ordinary reinforced masonry shear walls	2
ordinary plain masonry shear walls	1.5
Building Frame Systems	
ordinary steel concentrically braced frames	5
ordinary reinforced concrete shear walls	5
intermediate reinforced masonry shear walls	4
ordinary reinforced masonry shear walls	2.5
ordinary plain masonry shear walls	1.5
Moment Resistant Frame Systems	
intermediate steel moment frames	6
ordinary steel moment frames	4
intermediate reinforced concrete moment frames	5
ordinary reinforced concrete moment frames	3

Basic Seismic Force Resisting System	Response Modification Coefficient R
Bearing Wall Systems	
ordinary steel concentrically braced frames	4
ordinary reinforced concrete shear walls	4
intermediate reinforced masonry shear walls	3.5
ordinary reinforced masonry shear walls	2
ordinary plain masonry shear walls	1.5
Building Frame Systems	
ordinary steel concentrically braced frames	5
ordinary reinforced concrete shear walls	5
intermediate reinforced masonry shear walls	4
ordinary reinforced masonry shear walls	2.5
ordinary plain masonry shear walls	1.5
Moment Resistant Frame Systems	
intermediate steel moment frames	6
ordinary steel moment frames	4
intermediate reinforced concrete moment frames	5
ordinary reinforced concrete moment frames	3

Site Class

MCE-SRA at short periods (S_s) of 0.2 second and long periods (S_l) of 1.0 second are adjusted for Site Class and are thus designated (S_{Ms}) and (S_{Ml}).

$$S_{Ms} = F_a \, S_s$$

$$S_{Ml} = F_v \, S_l$$

where:

F_a = site coefficient (short period)
F_v = site coefficient (long period)

Design SPA

$$S_{Ds} = (2/3) \, S_{Ms}$$

$$S_{Dl} = (2/3) \, S_{Ml}$$

Design Response Spectrum

For T less than or equal to T_O:

$$S_a = 0.6\,(S_{Ds}/T_O)\,T + 0.4\,S_{Ds}$$

where $T_O = 0.2\,(S_{Dl}/S_{Ds})$

For T greater than or equal to T_O and less than or equal to T_S:

$$S_a = S_{Ds}$$

where: $T_S = S_{Dl}/S_{Ds}$

For T greater than T_S:

$$S_a = S_{Dl}/T$$

Category (A)

- provide a complete lateral-force-resisting system

- apply lateral force F_x at each floor level ($F_x = 0.01\ w_x$) where w_x is the portion of the gravity load (W) assigned to level x

- interconnect all parts of the structure

- anchor all concrete and masonry walls (the building is considered to be fixed at its base)

Simplified Analysis

(Seismic Use Group I, light frame construction, not more than 3 storeys *or* any form of construction not exceeding 2 storeys.)

- Seismic base shear

$$V = 1.2\,(S_{DS}/R)\,W$$

where R is the Response Modification Factor

- Lateral force

$$F_x = 1.2\,(S_{DS}/R)\,w_x$$

where w_x is the portion of the gravity load (W) assigned to level x

- assume a design storey drift of 1%

- anchor all concrete and masonry walls

Equivalent Lateral Force Procedure

(Seismic Categories B and C)

$$V = C_s W$$

where: C_s = the seismic response coefficient

$$C_s = S_{Ds}/(R/I_E)$$

where: I_E = Seismic Occupancy Importance Factor (1.0 to 1.5)

C_S need not exceed: $S_{Dl}/\{(R/I_E)\ T\}$

approximate T (T_a) may be obtained from:

$$T_a = C_T\, h_n^{0.75}$$

where:

C_T = the Building Period Coefficient (0.035 to 0.020)
h_n = height in feet of the building

Alternatively:

$$T_a = 0.1\ N$$

where:

N = number of storeys

P-Delta effects on storey shears and moments, resulting member forces and moments, and storey drifts induced by these effects are not required to be considered when the stability coefficient (θ) is not greater than 0.10 as determined by:

$$\theta = P_x \Delta / (V_x h_{sx} C_d)$$

where:

P_x = the total unfactored design load at and above level x
Δ = design storey drift occurring with V_x
V_x = the seismic shear force acting between levels x and x-1
h_{sx} = the storey height below level x
C_d = the deflexion amplification factor in NEHRP