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SOME NOVEL ASPECTS OF A SEISMIC CODE PROPOSAL FOR THE DOMINICAN REPUBLIC.

Dionisio BERNAL[1], Hector O'REILLY[2], Leonardo REYES MADERA[3] and Daniel COMARAZAMY[4]

SUMMARY

A draft of a new seismic code for the Dominican Republic is currently under review. The document follows the general philosophy of modern provisions such as NEHRP and IBC-2000 but contains some novel aspects. Two that are particularly noteworthy are: 1) the treatment of accidental eccentricity by means of an explicit torsional spectrum and 2) provisions for an explicit check of instability during inelastic seismic response of structures where all seismic loads are carried by moment frames.

INTRODUCTION

With a surface area of around 48,000 km², the Dominican Republic shares the island of Hispaniola with Haiti. The island has maximum dimensions of approximately 250 and 550 km in the north-south and east-west direction respectively and, as will be described in a subsequent section, is located in a region of the globe that has significant seismicity. Guidelines for seismic design in the country were introduced in 1976. In the year 2000, as part of an effort to update the countries construction codes, a project to formulate a new set of seismic design provisions was formulated. The document that was prepared, which is currently under review by the pertinent authorities, follows the general philosophy of modern seismic codes such as the NEHRP Provisions and the IBC-2000 code but is not a direct adaptation of any particular document. Two novel features of the proposed draft provisions are: 1) the use of a rotational base excitation as an alternative for achieving the objectives typically assigned to accidental eccentricity and 2) the inclusion of explicit guidelines for checking the safety against dynamic instability.

1 Dionisio Bernal, Associate Professor, Northeastern University, Boston MA
2 Hector O'Reilly, SODOSISMICA, Santo Domingo, Dominican Republic
3 Leonardo Reyes Madera, Consultant, Santo Domingo, Dominican Republic
4 Daniel Comarazamy, INTEC, Santo Domingo, Dominican Republic

This paper gives an overview of the basic framework of the draft provisions, which we shall refer to as the DP (Draft Provisions), and provides a detailed description of the novel aspects outlined previously. A section on the seismicity of the island, with emphasis on the eastern part occupied by the Dominican Republic, is presented prior to the discussing the design guidelines.

SEISMICITY

The island of Hispaniola has been historically affected by at least one major earthquake in every century since its discovery by the Spaniards in 1492. A map showing the historical seismicity is depicted in fig.1

Fig. 1 Epicenters and dates of past earthquakes

The fundamental source of the seismicity of the DR is the location of the island in the north of the Caribbean Plate, where this plate is in contact and interacts with the North American Plate. Evidence for the accumulation of stress along the interface between these plates is provided by recent GPS measurements which show that the Caribbean Plate is moving, relative to the North American Plate, in an east-northeast (070º) direction, approximately 18-20 mm/year. The main faults affecting the seismicity of the DR are depicted in fig.2.

A tectonic model of the Hispaniola, presented by Paul Mann and others [1,2] utilized measurements from 16 GS stations collected between 1994 and 1999 and has offered a partition of motion between the various faults. Computations based on the estimated slip and the time through which strains have accumulated (approximately 800 years) puts the maximum magnitude for the Septentrional fault at around 7.8 (moment magnitude) (Calais [3,4]).

The most recent earthquake of significant magnitude registered in the Dominican Republic occurred September 22, 2003, this event had a moment magnitude of 6.5 and was produced by the North Hispaniola fault. Two deaths occurred as a result of the earthquake.

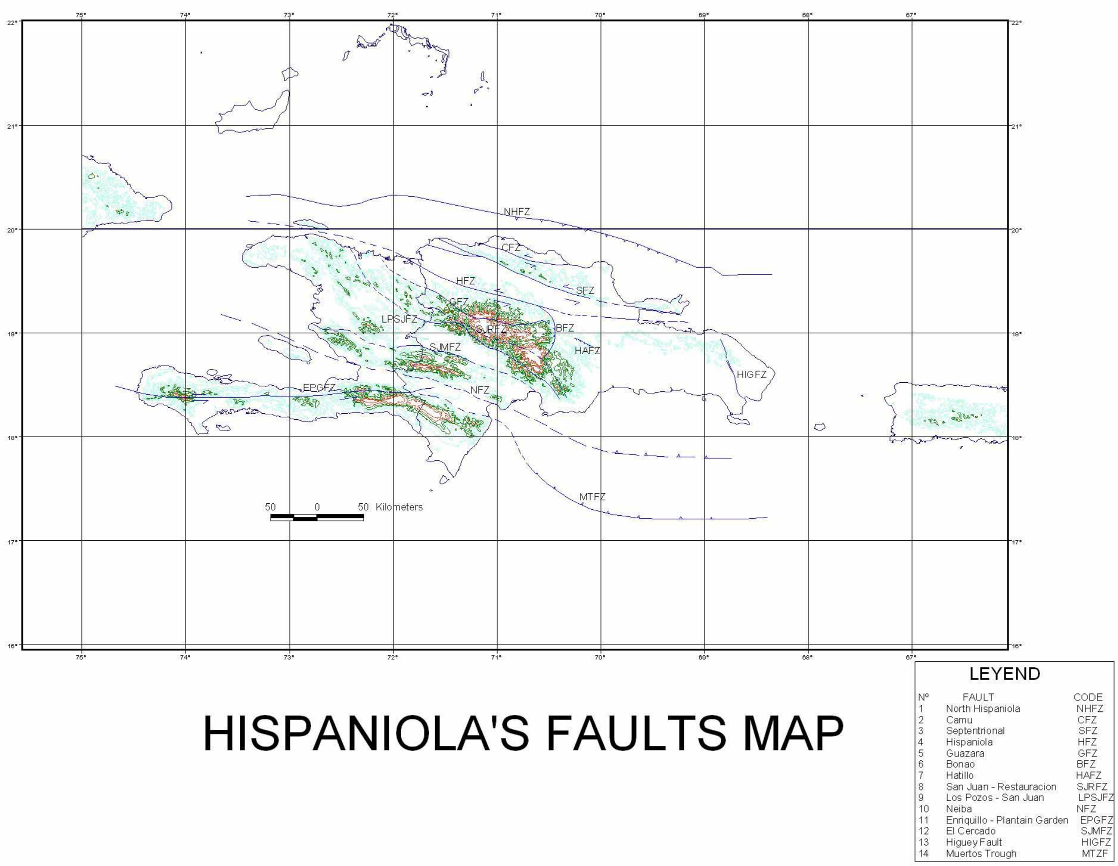

Fig.2 Main Faults affecting the Seismicity of the Dominican Republic

AN OVERVIEW OF THE DRAFT PROVISIONS

Definition of the Excitation

The DP defines the ground motion in a manner entirely analogous to that used by the NEHRP guidelines or the IBC-2000 code. Specifically, an elastic spectrum for 2% probability of being exceeded in 50 yrs is defined and the design ordinates are taken as 2/3 of the associated values. The 2% probability in 50yrs corresponds to a ground motion with a recurrence period of around 2500 yrs, which is satisfyingly long for a collapse level motion. The 2/3 factor applied to the elastic spectral ordinates is based on a contention that the ground motion needed to initiate collapse is approximately 50% larger than that required to drive the system to the inelastic design limit state addressed explicitly in the DP. In line with IBC-2000, the design spectrum is parameterized in terms of spectral ordinates S_s and S_1 associated with periods of 0.2 sec and 1.0

sec respectively. The spectral ordinates for 0.2 and 1.0 second period were computed using all the historical and instrumental information available using standard probabilistic seismic risk procedures. Results were obtained for a grid 20 km x 20 km covering the entire territory and were used to prepare the contour plots that are shown in fig 3. The design spectrum is defined as;

$$S_a = 0.6\frac{S_{DS}}{T_0}T + 0.4S_{DS} \qquad (T \leq T_0) \tag{1}$$

$$S_a = S_{DS} \qquad (T_0 \leq T \leq T_s) \tag{2}$$

$$S_a = \frac{S_{D1}}{T} \qquad T_0 > T \tag{3}$$

where

$$S_{DS} = \frac{2}{3}F_a S_s \tag{4}$$

$$S_{D1} = \frac{2}{3}F_v S_1 \tag{5}$$

The factors F_a and F_v account for local soil effects (as a function of the intensity of the motion).

Fig.3a – Contours of S_s

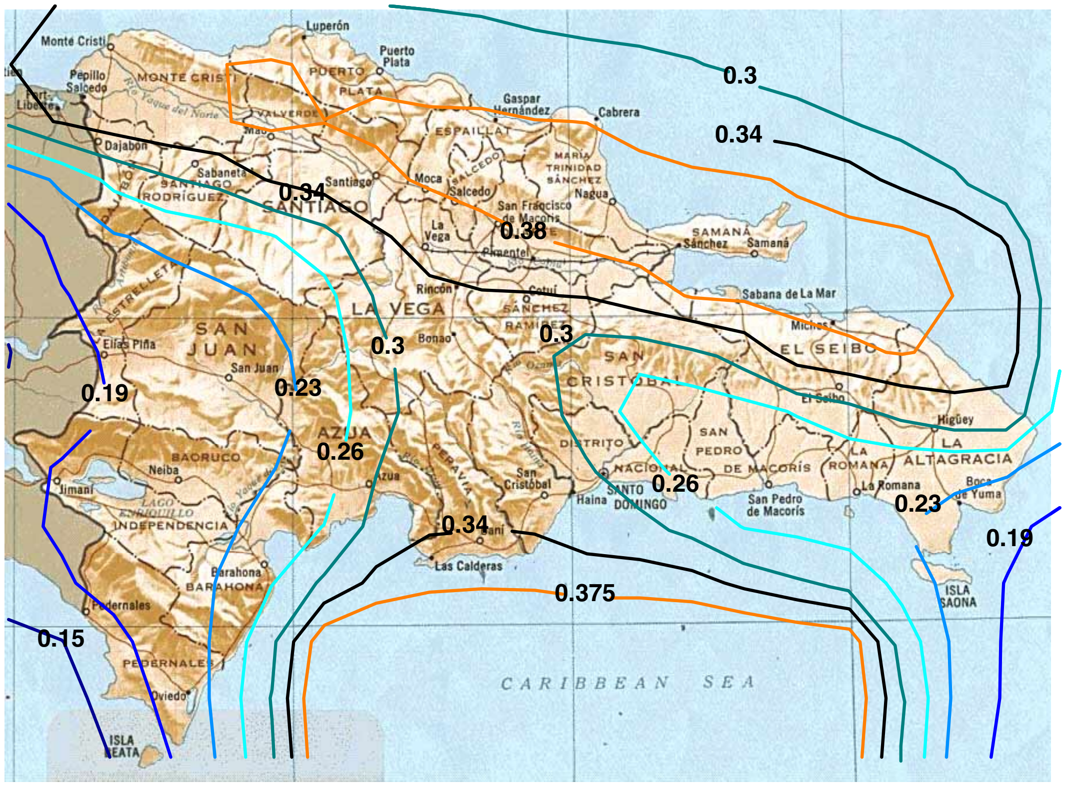

Fig.3b Contours of S_1

The periods defining the edges of three sections of the spectrum are;

$$T_0 = 0.2 \frac{S_{D1}}{S_{DS}} \tag{6}$$

$$T_S = 5T_0 \tag{7}$$

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and the factors F_a and F_v account for local soil effects (as a function of the intensity of the motion). The possibility of using time history records to define the design motion is permitted and guidelines on how to do it are provided.

Analysis Methods
The DP follow a conventional approach regarding ways to compute the effects of the motion for design purposes. Specifically, a static analysis based on equivalent lateral forces and dynamic response spectrum based analyses are provided as principal techniques. The possibility of carrying out nonlinear dynamic analysis to justify the adequacy of a particular design is also contemplated.

NOVEL ASPECTS

Two novel features incorporated into the DP are: 1) use of a torsional spectrum as an alternative to the accidental eccentricity provisions when performing 3D modal analysis and 2) explicit provisions to account for dynamic instability in buildings where all the seismic forces are resisted by frames.

Accidental Eccentricity - Torsional Spectrum
Accidental eccentricity provisions are intended to provide a lower bound to the torsional strength of structures and have been a part of seismic codes for several decades. The need for these provisions is evident when one recognizes that structures that are nominally symmetric and are analyzed for translational earthquake input display pure translational response. In these cases potential vulnerability to a twisting failure mode can go undetected because the torsional excitation is zero. Needless to say, torsional response occurs in all cases and it is essential that structures be designed to have a minimum strength against moments about the vertical axis. The provisions for accidental eccentricity attain this objective by requiring that buildings be able to resist the seismic forces acting at positions displayed from the center of mass by some particular amount. Although the provisions for accidental eccentricity are easily applied when the analysis is carried out for static forces, difficulties arise when seismic effects are computed using a response spectrum and modal analysis techniques.

Among the ad hoc procedures used by designers to incorporate ‘accidental eccentricity’ in dynamic analysis the most common is to examine the response of a sequence of models obtained by shifting all the centers of mass. Apart from the fact that the changes in dynamic properties that result when all the masses are shifted in a given direction are difficult to justify, from a pure practical perspective the consideration of accidental eccentricity in this manner is cumbersome, since it doubles the number of dynamic analyzes required. An informal survey carried out by the first author in the DR showed that designers that compute seismic effects from a dynamic analysis of a 3D model often discard accidental eccentricity provisions, sometimes under the misguided impression that the dynamic nature of the analysis eliminates the need for this requirement.

A natural solution for imposing a lower bound on torsional strength within the framework of 3-D dynamic analysis is by prescribing a torsional base input. Indeed, a torsional input can be treated as an additional spectrum and combined with the results of the translational spectra in a rational and consistent fashion. By appropriately selecting the amplitude of the torsional spectrum it is possible to simulate the accidental eccentricity loading.

In the DP the torsional spectrum is specified as being equal to the translational spectrum multiplied by a factor β that is given by;

$$\beta = 0.025 \frac{1}{N} \sum_{i=1}^{N} \frac{\bar{b}_i}{\tilde{r}_i^{\,2}} \tag{8}$$

where;

$\bar{b}_i$ = average of the two dimensions of the smallest rectangle that completely contains the plan of level i.

$\tilde{r}_i$ = radius of gyration of level i (square root of the ration between the rotational inertia and the mass).

N = Number of floors.

If the spectra for the two directions of analysis differ due to changes in the coefficient that accounts for inelastic behavior the torsional spectrum is defined using the larger one of the two (in other words, there is a single torsional spectrum, not one associated with each direction of analysis). The introduction of a torsional spectrum requires that one specify the point at the base where the rotational component of the motion is applied; in the DP this point is taken where a vertical axis passing through the center of mass of the complete structure intersects the base.

The coefficient in eq.8 can be rationalized as follows. The inertial loads due to the action of one component of base motion for mode i in a multistory structure (with rigid floor diaphragms) can be written as;

$$\begin{Bmatrix} f_x \\ f_y \\ f_z \end{Bmatrix}_i = \Gamma_i \, S_i \begin{bmatrix} M & & \\ & M & \\ & & J \end{bmatrix} \begin{Bmatrix} \phi_x \\ \phi_y \\ \phi_z \end{Bmatrix}_i \tag{9}$$

where the coordinate system has been assumed located at the center of mass of each level and the z subscript refers to twists about a vertical axis (i.e., f_z are torques and ϕ_z are rotations); S_i is the ordinate of the acceleration spectrum at the period of mode i and Γ_i is given by;

$$\Gamma_i = \begin{bmatrix} \phi_x^T & \phi_y^T & \phi_z^T \end{bmatrix}_i \begin{bmatrix} M & & \\ & M & \\ & & J \end{bmatrix} \begin{Bmatrix} r_x \\ r_y \\ r_z \end{Bmatrix} \tag{10}$$

where we've assumed that the modes have been mass normalized. The vector on the extreme right in eq.9 identifies the direction of the base excitation, for example, for translational inputs in x-x r_x ={1} and r_y = r_z = {0}, (vectors of appropriate dimension). If one assumes that the structure has a symmetric stiffness distribution (the case where protection from the accidental eccentricity provisions are most likely needed) the torsional and translational modes become uncoupled, which in the context of eq.9 means that only one of the partitions of the mode shape vector is non-zero in any given mode. Consider the two fundamental modes in translation and the fundamental mode in torsion, with the previous notation one can write;

$$f_x = \Gamma_x S_x M \phi_x \tag{11a}$$

$$f_y = \Gamma_y S_y M \phi_y \tag{11b}$$

$$f_z = \Gamma_z S_z J \phi_z \tag{12}$$

Assume now that we wish to specify the torques in eq.12 as the product of some eccentricity times the loads in eq.11. Assume that the x-x direction is selected as a reference (it will be apparent after completing the arguments that the selection is immaterial in the current context). Listing the eccentricities in a diagonal matrix E one can write;

$$S_z J \phi_z = \frac{\Gamma_x}{\Gamma_z} S_x E M \phi_x \tag{13}$$

expressing the rotational inertia matrix as a product of the mass times the square of the radius of gyration and recognizing that E and M are both diagonal (so the order of the product can be reversed) one gets;

$$E\phi_x = \frac{\Gamma_z S_z}{\Gamma_x S_x} [\tilde{r}^2] \phi_z \tag{14}$$

which shows that the eccentricity in level j is;

$$e_j = \frac{\Gamma_z S_z}{\Gamma_x S_x} \tilde{r}_j^2 \left(\frac{\phi_z}{\phi_x} \right)_j \tag{15}$$

A point to note at this juncture is that the periods at which S_z and S_x are evaluated differ, defining;

$$\gamma = \frac{S_x (at\, T = T_x)}{S_x (at\, T = T_z)} \tag{16}$$

and

$$S_z = \beta S_x \tag{17}$$

one gets;

$$e_j = \frac{\Gamma_z \beta}{\Gamma_x \gamma} \tilde{r}_j^2 \left(\frac{\phi_z}{\phi_x} \right)_j \tag{18}$$

While the traditional approach to specify accidental eccentricity uses values that depend on the dimension of the floor plan that is normal to the direction of analysis, the justification for this approach, given the objective of providing a lower bound to torsional strength is questionable. In the DP the eccentricity used to fix the lower bound torsional strength (when the torsional spectrum alternative is used) is based on the average of the two dimensions of the smallest rectangle that contains the floor plan, taking the eccentricity e_j as 0.05 times this dimension one gets;

$$\beta = 0.05 \bar{b}_j \frac{\Gamma_x \gamma}{\Gamma_z \tilde{r}_j^2} \left(\frac{\phi_x}{\phi_z} \right)_j \tag{19}$$

the value of β of eq.19 is not necessarily identical when computed for every level but, for a wide range of structures the value varies little (indicating that the mass normalized uncoupled modes for torsion and translation are similar). To pass from eq.19 to the simpler form given in the DP γ was taken as 0.5 (since the fundamental torsional period is generally significantly smaller than the translational ones), the ratio of the average dimension to the square of the radius of gyration (which may vary from floor to floor) was replaced by the average in the building and the other ratios (which can be shown to be equal to 1 for buildings with a repeated floor plan) where taken as one.

Dynamic Instability Check

Collapse from seismic excitation can occur because the ductile capacity of a system is exceeded and the strength degrades, or can be precipitated (prior to any material related distress) by instability from second order effects. The phenomenon of dynamic instability can occur when the fundamental buckling eigenvalue is reduced below unity during the inelastic dynamic response (fig.4). It's important to emphasize that the existence of a buckling eigenvalue less than unity during the inelastic response is necessary but not sufficient for instability to occur. In particular, for sufficiently small segments of time the structure can survive configurations that are statically unstable thanks to stabilizing inertial effects. Studies have shown that a critical item needed to assess the vulnerability against instability is the shape of the mechanism that controls the failure. As one anticipates, the structures that are most vulnerable to dynamic instability are those where moment frames resist all the lateral forces because in this case the critical mechanism can involve drift of only a few floors (Bernal [5], [6]).

The DP offers two methods to ensure that the safety margin against instability is adequate, in the first one the critical mechanism is explicitly estimated using a push-over analysis while the second approach conservatively assumes that the controlling mechanism involves drift in one single story (all stories are considered as possibilities); only structures that resist all lateral forces with moment frames require the explicit check. The guidelines are summarized next, the analytical support can be found in the previously mentioned references by Bernal.

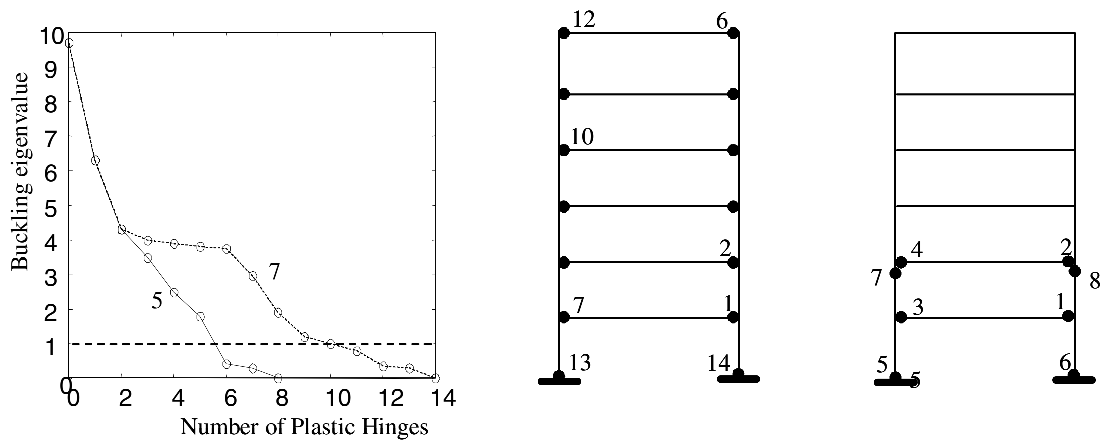

Fig.4 Effect of the distribution of inelasticity on the buckling eigenvalue.

*Detailed method***.**
Safety against dynamic instability can be assumed satisfied if;

$$V_u \geq 2V_c \tag{20}$$

where;

V_u = base shear at the formation of a mechanism. This base shear can be computed using a push-over analysis where the lateral load distribution is taken proportional to the story weights. Interaction between axial force and moment in the yield surface of columns must be considered.

V_c = base shear capacity at which incipient instability is anticipated, it is computed as;

$$V_c = 1.2 \frac{S_1}{g} \left(\frac{\Omega}{h} \right)^{0.75} W \tag{21}$$

where;

Ω = a factor given by;

$$\Omega = \frac{1 + 2N(1 - E/h - 0.5G/h)}{G/h[1 + 2N(1 - E/h - 0.67G/h] + \dfrac{1}{3N}} \tag{22}$$

where;

N = number of levels above the base.

E *y* G = distances defining the form of the critical mechanism (fig.5)

h = total height.

S_1 = elastic spectral ordinate at T= 1 seg (fig.3b)

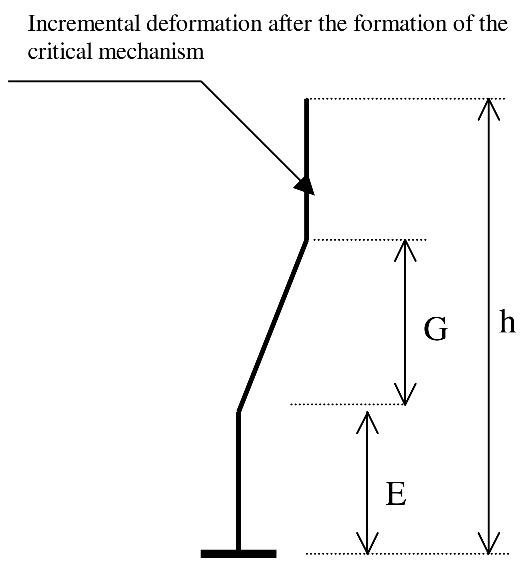

Fig.5 Shape of critical mechanism

*Simplified method***.**
The push-over analysis of the previous section can be bypassed if, at every level, and in each of the two directions of analysis the following inequality is satisfied;

$$V_{n,j} \geq 1.25 V_{c,j} \tag{23}$$

where;

$$V_{n,j} = \sum_{i=1}^{nc} \frac{(m_{n,t} + m_{n,b})}{\tilde{h}_j} \tag{24}$$

$$V_{c,j} = 1.2 \frac{S_1}{g} \left(\frac{1}{\tilde{h}_j} \right)^{0.75} \sum_{i=N}^{i=j} w_i \tag{25}$$

in these expressions;

$\tilde{h}_j$ = height of interstory j (in meters).

$m_{n,t}$ y $m_{n,b}$ = resisting moments at the column ends (without ϕ factor reduction). These strengths should be computed for an axial force level corresponding to 1.15 times that induced by the dead load.

w_i = weight of level i.

CONCLUSIONS

The new draft of seismic provisions for the Dominican Republic has been prepared after a careful review of the latest versions of the SEAOC blue book, the NEHRP provisions and the IBC-2000. The draft is not, however, an adaptation of any particular documents and contains some novel provisions. The two that are most significant are the option to treat accidental eccentricity through a torsional spectrum when the seismic effects are computed through the modal analysis of a 3D model and the provisions to check for safety against dynamic instability explicitly. Other provisions that are unique to this draft, not described in the body include: a) specifications regarding the stiffening effect of finite joint sizes, b) expressions to discriminate when the correlation between closely spaced modes has to be considered and when it doesn't and c) alternative ways to scale dynamic analysis results so that the base shear for the pseudo-static case doesn't have to be computed explicitly.

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