ΠΑΡΑΤΗΜΑ Α

Κανονιστικές Διατάξεις που αφορούν την Κατακόρυφη Συνιστώσα του Σεισμού.

1. ΕΑΚ 2000
2. Ευρωκώδικα 8 Μέρος 1 – Γενικά και κτίρια
3. Ευρωκώδικα 8 Μέρος 2 – Γέφυρες
4. Οδηγίες ΥΠΕΧΩΔΕ για τον Αντισεισμικό Σχεδιασμό Γεφύρων
5. SEAOC (Σεισμολογικά)
6. SEAOC (Υπολογιστικά)
7. International Building Code 2000
8. Ευρωκώδικα 8 Μέρος 1 – Γενικά και κτίρια (Σχέδιο τροποποίησης που εγκρίθηκε)
9. CALTRANS
10. Κανονιστικές Προτάσεις (Button et al.)
11. AASHTO (Σεισμική μόνωση)

EAK 2000

[3] Αν δεν υπολογίζεται η ιδιοπερίοδος Τ, τότε το $\Phi_d(T)$ θα λαμβάνεται από την εξίσωση *(2.1.β)*.

2

[4] Σε κάθε περίπτωση απαιτείται:

$$\frac{\Phi_d(T)}{A\gamma_i} \geq 0.25 \quad (2.3)$$

2.3.2 Κατακόρυφη συνιστώσα

[1] Το φάσμα της κατακόρυφης συνιστώσας καθορίζεται από τις εξισώσεις (2.1) με τις εξής μεταβολές:

- αντί της οριζόντιας εδαφικής επιτάχυνσης Α χρησιμοποιείται η αντίστοιχη κατακόρυφη συνιστώσα $A_v = 0.70 \cdot A$,
- αντί του συντελεστή συμπεριφοράς q χρησιμοποιείται ο συντελεστής $q_v = 0.50q \geq 1.00$ και
- η τιμή του συντελεστή θεμελίωσης θ λαμβάνεται πάντοτε ίση με 1.0.

2.3.3 Σεισμική επιτάχυνση εδάφους

[1] Για την εφαρμογή του παρόντος Κανονισμού η Χώρα υποδιαιρείται σε τέσσερις Ζώνες Σεισμικής Επικινδυνότητας I, II, III και IV, τα όρια των οποίων καθορίζονται στον Χάρτη Σεισμικής Επικινδυνότητας της Ελλάδος *(Σχήμα 2.2)*.

[2] Στον *Πίνακα 2.1* δίνεται κατάλογος οικισμών του ελληνικού χώρου και η Ζώνη Σεισμικής Επικινδυνότητας στην οποία ανήκουν.

[3] Σε κάθε Ζώνη Σεισμικής Επικινδυνότητας αντιστοιχεί μία τιμή σεισμικής επιτάχυνσης εδάφους Α, σύμφωνα με τον *Πίνακα 2.2*.

[4] Οι τιμές των σεισμικών επιταχύνσεων εδάφους του *Πίνακα 2.2* εκτιμάται, σύμφωνα με τα σεισμολογικά δεδομένα, ότι έχουν πιθανότητα υπέρβασης 10% στα 50 χρόνια.

2.3.4 Συντελεστής σπουδαιότητας κτιρίων

[1] Τα κτίρια κατατάσσονται σε τέσσερις κατηγορίες σπουδαιότητας, ανάλογα με τον κίνδυνο που συνεπάγεται για τον άνθρωπο και τις κοινωνικοοικονομικές συνέπειες που μπορεί να έχει ενδεχόμενη καταστροφή τους ή διακοπή της λειτουργίας τους.

[2] Σε κάθε κατηγορία σπουδαιότητας αντιστοιχεί μία τιμή του συντελεστή σπουδαιότητας γ_I σύμφωνα με τον *Πίνακα 2.3*.

1

[3] Για τον υπολογισμό των πραγματικών (μετελαστικών) μετακινήσεων του συστήματος, οι μετακινήσεις που προκύπτουν από τον γραμμικό υπολογισμό με την σεισμική δράση σχεδιασμού θα πολλαπλασιάζονται επί τον αντίστοιχο συντελεστή συμπεριφοράς q.

[4] Οι δύο οριζόντιες και κάθετες μεταξύ τους συνιστώσες του σεισμού μπορεί να έχουν οποιοδήποτε προσανατολισμό ως προς την κατασκευή.

[5] Επιτρέπεται, γενικά, η παράλειψη της κατακόρυφης συνιστώσας του σεισμού, εκτός από τις περιπτώσεις φορέων από προεντεταμένο σκυρόδεμα και δοκών που φέρουν φυτευτά υποστυλώματα στις ζώνες σεισμικής επικινδυνότητας III και IV. Στις περιπτώσεις αυτές επιτρέπεται η προσομοίωση και ανάλυση των παραπάνω δομικών στοιχείων σύμφωνα με την *παρ. 3.6*, ανεξάρτητα από την υπόλοιπη κατασκευή. Επίσης, σε κτίρια από φέρουσα τοιχοποιία, θα πρέπει να διερευνάται, γενικά, η επίδραση της κατακόρυφης συνιστώσας του σεισμού.

3.1.2 Μέθοδοι υπολογισμού

[1] Προβλέπεται η εφαρμογή των παρακάτω δύο μεθόδων γραμμικού υπολογισμού της σεισμικής απόκρισης:

α) Δυναμική φασματική μέθοδος.

β) Απλοποιημένη φασματική μέθοδος (Ισοδύναμη στατική μέθοδος).

Το πεδίο και ο τρόπος εφαρμογής των δύο αυτών μεθόδων καθορίζονται στις *παρ. 3.4 και 3.5* αντίστοιχα.

[2] Σε εντελώς ειδικές περιπτώσεις επιτρέπεται, συμπληρωματικά προς τις παραπάνω μεθόδους, η εφαρμογή άλλων δοκίμων μεθόδων υπολογισμού, όπως γραμμική ή μη γραμμική ανάλυση με εν χρόνω ολοκλήρωση επιταχυνσιογραφημάτων, κλπ. Οι μέθοδοι αυτές θα εφαρμόζονται υπό μορφή πρόσθετων ελέγχων και προς την πλευρά της ασφάλειας.

[3] Στην περίπτωση των κτιρίων για την εφαρμογή οποιασδήποτε μεθόδου υπολογισμού χρησιμοποιείται, γενικά, χωρικό προσομοίωμα της κατασκευής.

[3] Επισημαίνεται ότι οι πραγματικές ανελαστικές μετακινήσεις του συστήματος είναι ανεξάρτητες από τον συντελεστή συμπεριφοράς q, διότι θεωρούνται ίσες με τις μετακινήσεις του απεριόριστα ελαστικού συστήματος. Κατά συνέπεια, οι μετακινήσεις Δ_e που προκύπτουν από τις δυνάμεις σχεδιασμού $F_e = F_e^* / q$ (δηλαδή από την εφαρμογή των φασμάτων σχεδιασμού των εξ. *2.1)* θα πρέπει να πολλαπλασιασθούν με το q για να δώσουν τις μετακινήσεις $\Delta_e^* = \Delta_u$ του απεριόριστα ελαστικού συστήματος.

[4] *Βλ. παρ. Σ.2.2.1.[2].*

[5] Η επιρροή της κατακόρυφης συνιστώσας του σεισμού θεωρείται ότι καλύπτεται, γενικά, από τους συντελεστές ασφάλειας $\gamma_g = 1.35$ και $\gamma_q = 1.50$ στο συνδυασμό βασικών δράσεων (χωρίς σεισμό), καθώς επίσης και από τα υφιστάμενα περιθώρια αξονικής αντοχής των κατακόρυφων στοιχείων. Ιδιαίτερη προσοχή απαιτείται στις περιπτώσεις κατά τις οποίες η υπόψη συνιστώσα προκαλεί μεταβολή των μηχανικών χαρακτηριστικών των δομικών στοιχείων λόγω εφελκυσμού (τοιχοποιίες, διάτμηση υποστυλωμάτων). Επίσης δυσμενής μπορεί να είναι η προς τα άνω δράση της κατακόρυφης συνιστώσας του σεισμού σε προεντεταμένες δοκούς.

Σ.3.1.2 Μέθοδοι υπολογισμού

[1] Η δυναμική φασματική μέθοδος περιλαμβάνει πλήρη ιδιομορφική ανάλυση του συστήματος, υπολογισμό της μέγιστης σεισμικής απόκρισης για κάθε ιδιομορφή ταλάντωσης και, τέλος, τετραγωνική επαλληλία των μέγιστων ιδιομορφικών αποκρίσεων.

Η απλοποιημένη φασματική μέθοδος δεν απαιτεί ιδιομορφική ανάλυση, στηρίζεται σε προσεγγιστική θεώρηση μόνον της θεμελιώδους ιδιομορφής ταλάντωσης, η οποία όμως "ενισχύεται" κατάλληλα ώστε τα προκύπτοντα αποτελέσματα να βρίσκονται προς την πλευρά της ασφάλειας. Για περισσότερες πληροφορίες σχετικά με τις δύο αυτές μεθόδους παραπέμπουμε στη σχετική βιβλιογραφία {2},{3},{4},{5}.

[2] Κατά την εφαρμογή των "χρονολογικών" μεθόδων η προκύπτουσα απόκριση είναι εξαιρετικά ευαίσθητη σε μικρομεταβολές των βασικών παραμέτρων του συστήματος (διέγερση, μάζα, δυσκαμψία, απόσβεση). Επίσης στην περίπτωση του μη-γραμμικού υπολογισμού απαιτείται προσεκτική προσομοίωση της ανακυκλικής συμπεριφοράς των πλαστικοποιούμενων περιοχών και εκ των προτέρων γνώση των διαστάσεων των διατομών και του οπλισμού (για κατασκευές από οπλισμένο σκυρόδεμα). Επομένως δεν πρόκειται για μεθόδους σχεδιασμού των φορέων, αλλά για μεθόδους ελέγχου της μετελαστικής συμπεριφοράς τους.

[3] Η προσφυγή σε χωρικό προσομοίωμα είναι αναπότρεπτη, ακόμα και για κτίρια με δύο άξονες συμμετρίας, λόγω της στρεπτικής επιπόνησης που εισάγει πάντοτε η τυχηματική εκκεντρότητα.

3.6 ΚΑΤΑΚΟΡΥΦΗ ΣΕΙΣΜΙΚΗ ΔΙΕΓΕΡΣΗ

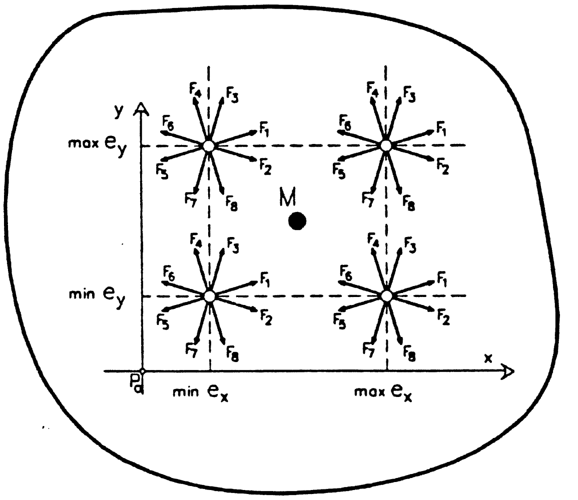
[1] Ο έλεγχος μεμονωμένων φορέων για κατακόρυφη σεισμική διέγερση μπορεί να γίνει με την απλοποιημένη φασματική μέθοδο ως ακολούθως:

α) Η κατακόρυφη σεισμική διέγερση εφαρμόζεται στα σημεία στήριξης του φορέα.

β) Η θεμελιώδης ιδιοπερίοδος του φορέα υπολογίζεται με τον τύπο του Rayleigh:

$$F_x = \begin{bmatrix} F_{1,x} \\ F_{2,x} \\ \cdot \\ \cdot \\ F_{N,x} \end{bmatrix}, \quad F_y = \begin{bmatrix} F_{1,y} \\ F_{2,y} \\ \cdot \\ \cdot \\ F_{N,y} \end{bmatrix}$$

Οι παραπάνω «ποσοστιαίοι συνδυασμοί» των στατικών φορτίσεων κατά x και y εφαρμόζονται διαδοχικά με τις μέγιστες και ελάχιστες εκκεντρότητες σχεδιασμού, οπότε προκύπτουν τελικά 4x8=32 περιπτώσεις στατικών φορτίσεων του κτιρίου *(σχ. Σ 3.5.3(4))*. Σε κάθε περίπτωση τα προκύπτοντα εντατικά μεγέθη επαλληλίζονται αλγεβρικά με τα αντίστοιχα εντατικά μεγέθη από τη δράση των κατακόρυφων φορτίων βαρύτητας.



Σχήμα Σ 3.5.3.(4): Ποσοστιαίοι συνδυασμοί οριζόντιων σεισμικών φορτίων.

Σ.3.6 ΚΑΤΑΚΟΡΥΦΗ ΣΕΙΣΜΙΚΗ ΔΙΕΓΕΡΣΗ

[1] Τα στηρίγματα των μεμονωμένων φορέων θεωρούνται κατακόρυφα. Σε περίπτωση κεκλιμένων στηριγμάτων θα έχουμε σύζευξη των κατακόρυφων και οριζόντιων ελευθεριών κίνησης στους κόμβους στήριξης, οπότε απαιτείται καθολική προσομοίωση του συστήματος και για την κατακόρυφη συνιστώσα του σεισμού.

$$T = 2\pi \cdot \sqrt{\frac{\sum_i m_i \cdot y_i^2}{\sum_i m_i \cdot y_i}} \quad \text{.......... (3.18)}$$

3

όπου y_i $(i = 1,2,...,n)$ οι μετατοπίσεις των συγκεντρωμένων μαζών m_i λόγω κατακόρυφων φορτίων $m_i \cdot 1$.

γ) Τα κατακόρυφα σεισμικά φορτία υπολογίζονται από τη σχέση:

$$F_i = M \cdot \Phi_{d.v}(T) \cdot \frac{m_i \cdot y_i}{\sum_j m_j \cdot y_j}, \quad (i, j = 1,2,..., n) \quad \text{.......... (3.19)}$$

όπου M η ταλαντούμενη μάζα του φορέα, $\Phi_{d.v}(T)$ η τιμή της φασματικής επιτάχυνσης σχεδιασμού και (n) ο αριθμός των συγκεντρωμένων μαζών m_i.

[2] Τα σεισμικά φορτία F_i εφαρμόζονται στατικά επάνω στον φορέα και η προκύπτουσα ένταση, τόσο του ίδιου όσο και των στοιχείων στήριξής του, προστίθεται στην ένταση από τις οριζόντιες συνιστώσες του σεισμού, αν δεν εφαρμοσθεί ακριβέστερη μορφή επαλληλίας.

[3] Η προηγούμενη μέθοδος επιτρέπεται να εφαρμόζεται ανεξάρτητα από την μέθοδο υπολογισμού για την οριζόντια σεισμική διέγερση.

3.7 ΠΡΟΣΑΡΤΗΜΑΤΑ ΚΤΙΡΙΩΝ

[1] Προσαρτήματα κτιρίων είναι κατασκευές ή τμήματα κατασκευών που δεν αποτελούν οργανικό μέρος του σκελετού όπως π.χ. στηθαία, καπνοδόχοι κλπ. Η σεισμική απόκριση ενός προσαρτήματος επηρεάζεται από την σεισμική απόκριση του κτιρίου επειδή η κίνηση του σημείου στήριξης πάνω στο κτίριο είναι διαφορετική από την κίνηση του εδάφους.

[2] Εάν δεν γίνεται ακριβέστερος υπολογισμός η οριζόντια σεισμική δύναμη για τον υπολογισμό των προσαρτημάτων και των στοιχείων στήριξής τους υπολογίζεται από την *εξίσωση (4.17)*, όπου ο σεισμικός συντελεστής ε δίδεται από την σχέση:

$$\varepsilon = \alpha \cdot \beta \cdot (1 + z / H) \quad \text{.......... (3.20)}$$

όπου:

$\alpha = A / g$,

[2,3] Ακριβέστερη μορφή επαλληλίας μπορεί να εφαρμοσθεί σύμφωνα με την *παρ. 3.5.3 ή την παρ. 3.4.4* σε περίπτωση εφαρμογής της δυναμικής φασματικής μεθόδου για την οριζόντια σεισμική δράση. Στην τελευταία αυτή περίπτωση ο τρίτος όρος στο δεύτερο μέλος της εξ. *(3.11.β)* αντικαθίσταται από τον τρίτο όρο του δεύτερου μέλους της εξ. *(3.17)*.

Σ.3.7 ΠΡΟΣΑΡΤΗΜΑΤΑ ΚΤΙΡΙΩΝ

[1] Εξωτερική διέγερση για το προσάρτημα αποτελεί η σεισμική απόκριση του σημείου στήριξης, λαμβανομένης υπόψη και της αλληλεπίδρασης κτιρίου-προσαρτήματος.

[2] Το προσάρτημα θεωρείται σαν μονοβάθμιο σύστημα, του οποίου η μέγιστη επιτάχυνση $\gamma = \varepsilon \cdot g$ λαμβάνεται ίση με το γινόμενο της επιτάχυνσης του κτιρίου $A \cdot (1 + z/H)$ στην στάθμη z επί τον συντελεστή αλληλεπίδρασης β μεταξύ προσαρτήματος-κτιρίου. Η θεωρούμενη εδώ επιτάχυνση κτιρίου (Α στη βάση και 2Α στην κορυφή) αποτελεί συντηρητική περιβάλλουσα των συνήθων επιταχύνσεων σχεδιασμού. Για $T_\pi > 2T$ λαμβάνεται $\beta = 1$ (αμελητέα αλληλεπίδραση).

Α.1 ΕΛΑΣΤΙΚΟ ΦΑΣΜΑ ΕΠΙΤΑΧΥΝΣΗΣ

[1] Οι οριζόντιες συνιστώσες των σεισμικών κινήσεων του εδάφους καθορίζονται με το επόμενο ελαστικό φάσμα επιτάχυνσης $\Phi_e(T)$:

$$0 \leq T < T_1 \qquad \Phi_e(T) = A\gamma_1\left[1+\left(\eta\beta_0 - 1\right)\frac{T}{T_1}\right]$$

$$T_1 \leq T \leq T_2 \qquad \Phi_e(T) = A\gamma_1\eta\beta_0$$

$$T_2 < T \qquad \Phi_e(T) = A\gamma_1\eta\beta_0\,\frac{T_2}{T}$$

όπου:

$\Phi_e(T)$	φασματική επιτάχυνση,
T	περίοδος σε δευτερόλεπτα,
T_1 και T_2	χαρακτηριστικές περίοδοι του φάσματος σε δευτερόλεπτα, οι οποίες δίδονται στον *Πίνακα 2.4* ανάλογα με την κατηγορία του εδάφους,
A	σεισμική επιτάχυνση του εδάφους κατά τον *Πίνακα 2.2,*
γ_1	συντελεστής σπουδαιότητας του κτιρίου κατά τον *Πίνακα 2.3,*
$\beta_0 = 2.50$	συντελεστής φασματικής ενίσχυσης και
η	διορθωτικός συντελεστής για ποσοστό κρίσιμης απόσβεσης διάφορο του 5%.

[2] Το ελαστικό φάσμα της κατακόρυφης συνιστώσας του σεισμού προκύπτει από το ανωτέρω ελαστικό φάσμα πολλαπλασιάζοντας τις τεταγμένες του με το 0.70.

[3] Σε περίπτωση αβεβαιότητας ως προς το έδαφος χρησιμοποιείται το δυσμενέστερο φάσμα.

Α.2 ΕΠΙΤΑΧΥΝΣΙΟΓΡΑΦΗΜΑΤΑ

[1] Επιτρέπεται η χρησιμοποίηση πραγματικών ή/και συνθετικών επιταχυνσιογραφημάτων, τα οποία στη συνέχεια του παρόντος Κανονισμού καλούνται «επιταχυνσιογραφήματα σχεδιασμού», εφόσον πληρούν τις διατάξεις της *παρ. Α.2.1.*

(2) EC8

Χενικα

(2) For most of the applications of this Eurocode, the hazard is described in terms of a single parameter, i.e. the value a_g of the effective peak ground acceleration in rock or firm soil, henceforth called "design ground acceleration". Additional parameters required for specific types of structures are given in the relevant Parts of Eurocode 8.

NOTE: The concept of the "effective peak ground acceleration" is an attempt to compensate for the inadequacy in general of the actual single peak to describe the damaging potential of the ground motion in terms of maximum acceleration and/or velocity induced to the structures.

There is not a unique established definition and corresponding techniques for deriving a_g from the ground motion characteristics, the methods actually varying as functions of these latter. In general, a_g tends to coincide with the actual peak for moderate-to-high magnitude of medium-to-long distance events, which are characterized (on firm ground) by a broad and approximately uniform frequency spectrum, while a_g will be more or less reduced relative to the actual peak for near field, low magnitude events.

(3) The design ground acceleration, chosen by the National Authorities for each seismic zone, corresponds to a reference return period of [475] years. To this reference return period an importance factor γ_I equal to 1,0 is assigned.

(4) Seismic zones with a design ground acceleration a_g not greater than [0,10]·g are low seismicity zones, for which reduced or simplified seismic design procedures for certain types or categories of structures may be used.

(5)P In seismic zones with a design ground acceleration a_g not greater than [0,04]·g the provisions of Eurocode 8 need not be observed.

4.2 Basic representation of the seismic action

4.2.1 General

(1)P Within the scope of Eurocode 8 the earthquake motion at a given point of the surface is generally represented by an elastic ground acceleration response spectrum, henceforth called "elastic response spectrum".

(2)P The horizontal seismic action is described by two orthogonal components considered as independent and represented by the same response spectrum.

(3) Unless specific studies indicate otherwise, the vertical component of the seismic action should be represented by the response spectrum as defined for the horizontal seismic action, but with the ordinates reduced as follows:

- For vibration periods T smaller than 0,15 s the ordinates are multiplied by a factor of [0,70].
- For vibration periods T greater than 0,50 s the ordinates are multiplied by a factor of [0,50].
- For vibration periods T between 0,15 s and 0,50 s a linear interpolation shall be used.

(4) For special conditions more than one spectrum may be needed to adequately represent the seismic hazard over an area. This may be necessary when the earthquakes affecting the area are generated by sources differing widely in distance, focal mechanism or travel path geology, as in the case of shallow depth and intermediate depth earthquakes. In such circumstances, different values of a_g as well as different shapes of the response spectrum for each type of earthquake would normally be required.

(5) For important structures in high seismicity zones it is recommended to consider topographic amplification effects according to Annex B of Part 5.

(6) Alternative representations of the earthquake motion - e.g. power spectrum or time history representation - may be used (see 4.3).

(7) Allowance for the variation of ground motion in space as well as time may be required for specific types of structures (see Parts 2, 3 and 4).

4.2.2 Elastic response spectrum

(1)P The elastic response spectrum $S_e(T)$ for the reference return period is defined by the following expressions (see figure 4.1):

$$0 \le T \le T_B: \qquad S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot \beta_o - 1)\right] \qquad (4.1)$$

$$T_B \le T \le T_C: \qquad S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_o \qquad (4.2)$$

$$T_C \le T \le T_D: \qquad S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_o \cdot \left[\frac{T_C}{T}\right]^{k_1} \qquad (4.3)$$

$$T_D \le T: \qquad S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_o \cdot \left[\frac{T_C}{T_D}\right]^{k_1} \cdot \left[\frac{T_D}{T}\right]^{k_2} \qquad (4.4)$$

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(2) The combination of the horizontal components of the seismic action may be accounted for as follows:

- The structural response to each horizontal component shall be evaluated separately, using the combination rules for modal responses as given in 3.3.3.2.
- The maximum value of each action effect on the structure due to the two horizontal components of the seismic action may then be estimated by the square root of the sum of the squared responses to each horizontal component.

(3) As an alternative to paragraph (2) the action effects due to the combination of the horizontal components of the seismic action may be computed using the two following combinations:

a) E_{Edx} "+" $0{,}30 \cdot E_{Edy}$

b) $0{,}30 \cdot E_{Edx}$ "+" E_{Edy}

where

"+" implies "to be combined with",

E_{Edx} action effects due to the application of the seismic action along the chosen horizontal axis x of the structure,

E_{Edy} action effects due to the application of the same seismic action along the orthogonal horizontal axis y of the structure.

(4) The sign of each component in the above combinations shall be taken as the most unfavourable for the effect under consideration.

(5)P For buildings satisfying the regularity criteria in plan and in which walls are the only horizontal load resisting components, the seismic action may be assumed to act separately along the two main orthogonal horizontal axes of the structure.

(6)P When using time-history analysis according to 3.3.4.3 and employing a spatial model of the structure, simultaneously acting accelerograms shall be considered for both horizontal components.

3.3.5.2 Vertical component of the seismic action

(1)P The vertical component of the seismic action, as defined in clause 4.2.1.(3) of Part 1-1, shall be taken into account in the following cases:

- Horizontal or nearly horizontal structural members spanning 20 meters or more;

- Horizontal or nearly horizontal cantilever components;

- Horizontal or nearly horizontal prestressed components;

- Beams supporting columns.

(2) In general, the analysis for determining the effects of the vertical component of the seismic action can be made based on a partial model of the structure which includes the elements under consideration and takes into account the stiffness of the adjacent elements.

(3) The effects of the vertical component need only be considered for the elements under consideration and their directly associated supporting elements or substructures.

(4) In case the horizontal components of the seismic action are also relevant for these elements, the following three combinations may be used for the computation of the action effects:

a) $0{,}30 \cdot E_{Edx}$ "+" $0{,}30 \cdot E_{Edy}$ "+" E_{Edz}

b) E_{Edx} "+" $0{,}30 \cdot E_{Edy}$ "+" $0{,}30 \cdot E_{Edz}$

c) $0{,}30 \cdot E_{Edx}$ "+" E_{Edy} "+" $0{,}30 \cdot E_{Edz}$

where

E_{Edx} see 3.3.5.1.(3),

E_{Edy} see 3.3.5.1.(3),

E_{Edz} action effects due to the application of the vertical component of the design seismic action as defined in clause 4.2.1.(3) of Part 1-1.

3.4 Displacement analysis

(1)P The displacements induced by the design seismic action shall be calculated on the basis of the elastic deformation of the structural system by means of the following simplified expression:

$$d_s = q_d \cdot d_e \cdot \gamma_I \tag{3.12}$$

where

υλικα

(9) Ο συντελεστής k_w που εκφράζει την επιρροή της κυρίαρχης μορφής αστοχίας σε δομικά συστήματα με τοιχώματα πρέπει να λαμβάνεται ως εξής:

$$k_w = \begin{cases} 1{,}00 & \text{για πλαισιακά και ισοδύναμα με πλαισιακά μικτά συστήματα} \\ 1/(2{,}5-0{,}5\cdot\alpha_o) \leq 1 & \text{για συστήματα τοιχωμάτων, συστήματα ισοδύναμα με συστήματα τοιχωμάτων και πυρήνες} \end{cases} \qquad (2.5)$$

όπου

α_o κυρίαρχη αναλογία διαστάσεων των τοιχωμάτων του στατικού συστήματος ($\alpha_o = (H_w/l_w)$).

(10) Εαν οι αναλογίες διαστάσεων H_{wi}/l_{wi} όλων των τοιχωμάτων i ενός στατικού συστήματος δεν διαφέρουν σημαντικά η κυρίαρχη αναλογία διαστάσεων α_o μπορεί να οριστεί ωσ εξής:,

$$\alpha_o = \Sigma H_{wi}/\Sigma l_{wi} \qquad (2.6)$$

όπου

H_{wi} ύψος του τοιχώματος i

l_{wi} μήκος της διατομής του τοιχώματος i

2.3.2.2 Κατακόρυφες σεισμικές δράσεις

(1) Για την κατακόρυφη συνιστώσα της σεισμικής δράσης, για όλα τα στατικά συστήματα πρέπει, γενικώς, να υιοθετείται συντελεστής συμπεριφοράς q ίσος με την μονάδα.

(2) Η αποδοχή τιμών του συντελεστή συμπεριφοράς q μεγαλυτέρων της μονάδας πρέπει να δικαιολογείται μέσω κατάλληλης ανάλυσης.

2.4 Κριτήρια σχεδιασμού

2.4.1 Γενικά

(1) Οι αρχές σχεδιασμού που περιγράφτηκαν στην 2.1.3 και στο μέρος 1-1 εφαρμόζονται για τα αντισεισμικά δομικά στοιχεία των κτιρίων από σκυρόδεμα όπως ορίζεται στις 2.4.2 ÷ 2.4.7.

(2) Τα κριτήρια σχεδιασμού που δίνονται στις 2.4.2 ÷ 2.4.7 θεωρείται ότι ικανοποιούνται, όταν οι διατάξεις που δίνονται στις 2.6 ÷ 2.12 τηρούνται.

2.4.2 Κριτήριο τοπικής αντοχής

(1) Ολες οι κρίσιμες περιοχές της κατασκευής πρέπει να έχουν αντοχή επαρκώς υψηλότερη των αντίστοιχων δράσεων που αναπτύσσονται σε αυτές τις περιοχές λόγω του σεισμού σχεδιασμού.

(2) Οι δευτερεύουσες δράσεις πρέπει να λαμβάνονται υπ'όψιν όπως περιγράφεται στο μέρος 1-2.

3. SEISMIC ACTION

3.1 Definition of the seismic action

3.1.1 General

(1)P The seismic action can be defined by means of different models, whose complexity shall be appropriate to the relevant earthquake motion to be described and the importance of the structure and commensurate with the sophistication of the model used for the idealisation of the bridge.

(2)P In this section only the shaking transmitted by the ground to the structure is considered in the quantification of the seismic action. However, earthquakes can induce permanent displacements in soils (ruptures, liquefaction of sandy layers and ground offset due to active faulting) that may result in imposed deformations with severe consequences to bridges. This type of hazard shall be evaluated through specific studies and its consequences shall be minimised by an appropriate selection of the structural foundation system and possibly ground improvement. Tsunami effects are not treated in this Code.

3.1.2 Seismological aspects

(1)P In the definition of the seismic action the following aspects shall be considered:

- the characterisation of the motion at a point;
- the characterisation of the spatial variability of the motion.

3.1.3 Application of the components of the motion

1(P) In general only the three translational components of the seismic action are taken into account. When the Response Spectrum method is applied the bridge may be analysed separately for shaking in the longitudinal, transverse and vertical directions. In this case the seismic action is represented by three one-component actions, one for each direction, quantified according to 3.2.2 and 3.2.3.2. The action effects shall be combined according to 4.2.1.4;

2(P) When linear time domain analysis is performed or when the six component model or the spatial variability of the seismic motion is taken into account, the bridge shall be analysed under the simultaneous action of the different components.

3.2 Characterisation of the motion at a point

3.2.1 General

(1)P The characterisation of the motion at a point shall be carried out in two phases:

- Quantification of each component of the motion;
- Construction of a three component model of the motion with three translational components, or of a six component model of the motion, with three translational components and three rotational components.

(2) The seismic action is applied at the interface between the structure and the ground. If springs are used to represent the soil stiffness either in connection with spread footings or with deep foundations (piles, shafts etc., see Part 5) the motion is applied at the soil end of the springs. In general it is not necessary to use the three rotational components of the ground motion. If their inclusion is considered necessary, then cl. 3.2.3 is applicable.

3.2.2 Quantifying of the components

3.2.2.1 General

(1)P Each component of the earthquake motion shall be quantified in terms of a response spectrum, or a power spectrum, or a time history representation (mutually consistent) as set out in Section 3 of Part 1, which also provides the basic definitions.

3.2.2.2 Site dependent elastic response spectrum

3.2.2.2.1 Horizontal component

(1)P The horizontal component shall be in accordance with 3.2.2.2 of Part 1.

3.2.2.2.2 Vertical component

(1)P When needed (see cl. 4.1.7), the site dependent response spectrum for the vertical component of the earthquake motion shall be taken in accordance with 3.2.2.of Part 1

3.2.2.2.3 Site averaged response spectrum

(1)P In the case of bridges whose abutments and piers are supported on soils having significantly different soil properties but which do not require the use of a spatial variability model for the seismic action, the site average response spectrum shall be defined by combining, through a validated scientific method, the spectra corresponding to the differing soil conditions of the supports.

(2) The site averaged response spectrum S_a may be defined as a weighted average of the appropriate site dependent response spectra and is determined by

$$S_a(T) = \sum_i \frac{r_i}{\sum_j r_j} S_i(T) \qquad (3.1)$$

where r_i is the reaction force on the base of pier i when the deck is subjected to a unit displacement while the base is kept immobile; S_i is the site dependent response spectrum appropriate to the soil conditions at the foundation of pier i.

Note: The average shall be computed separately for each of the two horizontal components and for the vertical component.

(3) Alternatively the site averaged response spectrum may be substituted by an envelope spectrum obtained by considering, for each period, the highest value of the site dependent response spectra corresponding to the different soil conditions at the foundations of the bridge.

3.2.2.3 Site dependent power spectrum

(1)P The earthquake action can be described by a stochastic stationary gaussian process defined by a power spectrum and considered with a duration limited to a given time interval. This description of the motion shall be consistent with the site dependent response spectrum. Consistency between power spectrum and response spectrum shall be defined as equality between the response spectrum value and the mean value of the probability distribution of the largest extreme value (for the duration considered) of the response of a one degree of freedom oscillator with a corresponding natural frequency and viscous damping.

Note: The term extreme value refers to the absolute value of a maximum or a minimum value. It should be noted that in some cases (local) maximum values may have negative values and (local) minimum values may have positive values.

3.2.2.4 Time history representation

(1)P When a non-linear time-history analysis is carried-out, at least three pairs of horizontal ground motion time-history components shall be used. The pairs should be selected from recorded events with magnitudes, source distances, and mechanisms consistent with those that define the design seismic action.

(2) When the required number of pairs of appropriate recorded ground motions is not available, appropriate simulated accelerograms may be used in replacement of the missing recorded motions.

(3)P Consistency to the applicable 5% damped design seismic spectrum shall be established by scaling the amplitude of motions as follows:

- For each earthquake consisting of a pair of horizontal motions the SRSS spectrum shall be created by taking the square root of the sum of squares of the 5%-damped spectra of each component.

- The spectrum of the ensemble of earthquakes shall be formed by taking the average value of the SRSS spectra of the individual earthquakes of the previous step.

- The ensemble spectrum shall be scaled so that it is not lower than 1.3 times the 5%-damped design seismic spectrum, in the period range between $0.2T_1$ and $1.5\ T_1$. Where T_1 is the natural period of the fundamental mode of the structure in the case of a ductile bridge, or the effective period (T_{eff}) of the isolation system in the case of a bridge with seismic isolation (see 7.2).

- The scale factor derived in the previous step shall be applied to all individual seismic motion components.

(4)P Each pair of time histories shall be applied simultaneously.

3.2.2.5 Site dependent design spectrum for linear analysis

(1)P Both ductile and limited ductile structures shall be designed by performing linear analysis using a reduced response spectrum called the design spectrum as specified by 3.2.2.5 of EN 1998-1, but the lower bound shall be taken equal to $S_d(T) > 0.10\alpha$

3.2.3 Six component model

3.2.3.1 General

(1)P The six component model of the earthquake motion at a point shall be developed from the probable contribution of the P, S, Rayleigh and Love waves to the total earthquake vibration.

> Note: The simplified models referred to in Annex D may be used if geological discontinuities are not present.

3.2.3.2 Separation of the components of the seismic action

(1)P For the separation of the components of the seismic action the relevant provisions of 3.1.3 are applicable. However, the vertical component may, in general, be disregarded if the bridge is not particularly sensitive to vibrations in this direction; furthermore, the rotational components are usually not important and can also be disregarded.

3.3 Characterisation of the spatial variability

(1)P The spatial variability shall be considered when:

- Geological discontinuities (e.g. soft soil contiguous to rock)
- Marked topographical features are present;
- The length of the bridge is greater than 600 m, even if there are no geological discontinuities or marked topographical features.

> Note: Simplified models to take into account the spatial variability of the earthquake motion are presented in Annex D.

(2)P The spatial variability dealt with in this subclause concerns the continuous deformation of the ground, in the elastic or in the post-elastic range. However, in the case of strong earthquakes, discontinuous deformations, due to surface faulting or soil ruptures, may be induced. Measures to prevent the risks related to this type of hazard, such as the adoption of structural systems which minimise its effects, shall be taken. (See also 2.4 (10)).

with the correlation factor

$$r_{ij} = \frac{8\xi'^2(1+\rho)\rho^{3/2}}{\left(1-\rho^2\right)^2 + 4\xi'^2\rho(1+\rho)^2} \tag{4.8}$$

where:
$\rho = T_j/T_i$ and
ξ is the viscous damping ratio

(4) When the differential displacement along the base of the bridge can induce substantial stresses in the structure, the value of the earthquake action effects can be determined in the case of application of the SRSS-rule as

$$E = \sqrt{\Sigma_i E_i^2 + \Sigma_m (k_m d_m)^2} \tag{4.9}$$

and in the case of application of the CQC-method as

$$E = \sqrt{\Sigma_i \Sigma_j E_i r_{ij} E_j + \Sigma_m (k_m d_m)^2} \tag{4.10}$$

where k_m is the effect of the m-th independent motion and d_m is the asymptotic value of the spectrum for the m-th motion for large periods, expressed in displacements.

4.2.1.4 Combination of the components of seismic action

(1) The probable maximum action effect E, due to the simultaneous occurrence of seismic actions along the horizontal axes X, Y and the vertical axis Z, may be estimated from the maximum action effects E_x, E_y and E_z due to independent seismic action along each axis, as follows:

$$E = \sqrt{E_x^2 + E_y^2 + E_z^2} \tag{4.11}$$

(2) Alternatively it is sufficient to use as design seismic action A_{Ed} the most adverse of the following combinations:

$$\begin{array}{l} A_{Ex} \text{ "+" } 0.30 A_{Ey} \text{ "+" } 0.30\, A_{Ez} \\ 0.30 A_{Ex} \text{ "+" } A_{Ey} \text{ "+" } 0.30 A_{Ez} \\ 0.30 A_{Ex} \text{ "+" } 0.30 A_{Ey} \text{ "+" } A_{Ez} \end{array} \tag{4.12}$$

where A_{Ex} , A_{Ey} and A_{Ez} are the seismic actions in each direction X, Y and Z respectively. A_{Ez} should be considered according to the requirements of 4.1.7.

4.2.2 Fundamental mode method

4.2.2.1 Definition

(1) Equivalent static seismic forces are derived from the inertia forces corresponding to the fundamental mode and natural period of the structure in the direction under consideration, using the relevant ordinate of the site dependant design spectrum. The method includes also simplifications regarding the shape of the first mode and the estimation of the fundamental period.

(2) Depending on the particular characteristics of the bridge, this method may be applied using three different approaches for the model, namely:

- the Rigid Deck Model
- the Flexible Deck Model
- the Individual Pier Model

(3)P The rules of 4.2.1.4 for the combination of the components of seismic action shall be applied.

4.2.2.2 Field of application

(1) The method may be applied in all cases in which the dynamic behaviour of the structure can be sufficiently approximated by a single dynamic degree of freedom model. This condition is considered to be satisfied in the following cases:

(a) In the longitudinal direction of approximately straight bridges with continuous deck, when the seismic forces are carried by piers whose total mass is less than 1/5 of the mass of the deck.

(b) In the transverse direction of case (a) when the structural system is approximately symmetrical about the centre of the deck, i.e. when the theoretical eccentricity e_o between the centre of stiffness of the supporting elements and the centre of mass of the deck does not exceed 5% of the length of the deck (L).

(c) In the case of piers carrying simply supported spans when no significant interaction between piers is expected and the total mass of each pier is less than 1/5 of the mass of the part of the deck carried by the pier.

4.2.2.3 Rigid deck model

(1) This model may be applied only when - under the earthquake action - the deformation of the deck in a horizontal plane is negligible compared to the displacements of the pier tops. This is always valid in the longitudinal direction of approximately straight bridges with continuous deck. In the transverse direction the deck may be assumed rigid if L/B ≤ 4.0 or, in general, if the following condition is satisfied:

$$\frac{\Delta_d}{d_a} \leq 0.20 \qquad (4.13)$$

Type of Ductile Elements	Seismic Behaviour	
	Limited Ductile	Ductile
Reinforced concrete piers:		
Vertical piers in bending ($\alpha_s^* \geq 3.0$)	1.5	3.5 $\lambda(\alpha_s)$
Inclined struts in bending	1.2	2.1 $\lambda(\alpha_s)$
Steel Piers:		
Vertical piers in bending	1.5	3.5
Inclined struts in bending	1.2	2.0
Piers with normal bracing	1.5	2.5
Piers with eccentric bracing		3.5
Abutments rigidly connected to the deck:		
In general	1.5	1.5
Locked in structures (par. (9), (10))	1.0	1.0
Arches	1.2	2.0

*α_s= L/h is the shear ratio of the pier, where L is the distance from the plastic hinge to the point of zero moment and h is the depth of the cross section in the direction of flexure of the plastic hinge.

For $\alpha_s \geq 3$ $\quad \lambda(\alpha_s) = 1.0$

$3 > \alpha_s \geq 1.0$ $\quad \lambda(\alpha_s) = \sqrt{\frac{\alpha_s}{3}}$

(4)P For reinforced concrete elements the values of q-factors given in Table 4.1 are valid when the normalised axial force η_k defined in 5.3 (4) does not exceed 0.30.

When $0.30 < \eta_k \leq 0.60$, even at a single ductile element, the value of the behaviour factor shall be reduced to:

$$q_r = q-(\eta_k/(.3)-1)(q-1) \geq 1.0 \qquad (4.2)$$

A value for $q_r = 1.0$ (elastic behaviour) should be used for bridges in which the seismic force resisting system contains elements with $\eta_k \geq 0.6$.

(5)P The values of the q-factor for Ductile Behaviour given in Table 4.1 may be used only if the locations of all the relevant plastic hinges are accessible for inspection and repair. Otherwise, the values of Table 4.1 shall be multiplied by 0.6; however final q-values less than 1.0 need not be used. When energy dissipation is intended to occur at plastic hinges located in piles, which are designed for ductile behaviour, and at points which are not accessible, a final q-value of 2.1 shall be used for vertical piles and 1.5 for inclined piles.

Note: The term "accessible", as used in the paragraph above, has the meaning of "accessible even with reasonable difficulty". The foot of a pier shaft located in backfill, even at substantial depth, is considered to be "accessible". On the contrary, the foot of pier shaft immersed in deep water, or the heads of piles beneath an extensive pile cap, should not be considered "accessible".

(6) Regarding plastic hinge formation in the deck, see 2.3.2.1 (4).

(7) No plastic hinges will in general develop in piers flexibly connected to the deck, in the direction under consideration. A similar situation will occur in individual piers having very low stiffness in comparison to the other piers (see 2.3.2.1 (6) and (7)). Such elements have negligible contribution in resisting the seismic actions, and therefore do not affect the q-factor (see 4.1.6 (3)P).

(8) When the bridge has a regular seismic behaviour, as defined in 4.1.8, the value of behaviour factor as defined above may be used without any additional check.

(9) Bridge structures whose mass follows essentially the horizontal seismic motion of the ground ("locked-in" structures), do not experience significant amplification of the horizontal ground acceleration. Such structures are characterised by a very low value of the natural period in the horizontal directions ($T \leq 0.03$ s). The inertial response of these structures in the horizontal directions may be assessed using the design value of the seismic ground acceleration and $q = 1$. Abutments flexibly connected to the deck belong to this category.

(10) Bridge structures consisting of an essentially horizontal deck, rigidly connected to both abutments (either monolithically or through fixed bearings or links), may be considered to belong to the category of the previous paragraph (9) (without need to check the natural period) if the abutments are laterally encased, at least over 80 % of their area, in stiff natural soil formations. If above conditions are not met, then the soil interaction at the abutments should be included in the model, using realistic soil stiffness parameters. In case $T > 0.03$ s, then the normal acceleration response spectrum with $q = 1.50$ should be used.

(11) When the main part of the design seismic action is resisted by elastomeric bearings the flexibility of the bearings imposes a practically elastic behaviour of the system, i.e. $q \cong 1.0$. Such bridges shall be designed according to Section 7. The potential formation of plastic hinges in secondary deck elements (continuity slabs) is allowed but should not be considered to increase the value of q.

(12)P The behaviour factor for the analysis in the vertical direction shall always be taken equal to 1.0.

4.1.7 Vertical component of the seismic action

(1) The effects of the vertical seismic component on the piers may, as a rule, be omitted in zones of low and moderate seismicity. In zones of high seismicity these effects need only be investigated in the exceptional cases when the piers are subjected to high bending stresses due to the permanent actions of the deck.

(2)P The effects of the vertical seismic component in the upward direction in prestressed concrete decks, shall be always investigated.

(3)P The effects of the vertical seismic component on bearings and links shall be assessed in all cases.

(4) The estimation of the effects of the vertical component may be carried out using the Fundamental Mode Method and the Flexible Deck Model (see 4.2.2.4).

A

ΟΔΗΓΙΕΣ

ΓΙΑ ΤΗΝ ΑΝΤΙΣΕΙΣΜΙΚΗ ΜΕΛΕΤΗ ΓΕΦΥΡΩΝ

ΚΕΙΜΕΝΟ & ΣΧΟΛΙΑ

Νοέμβριος 1999

2. ΜΕΘΟΔΟΣ ΦΑΣΜΑΤΙΚΗΣ ΑΠΟΚΡΙΣΗΣ

2.1 Φάσμα Επιταχύνσεων Σχεδιασμού

(1) Το φάσμα οριζοντίων επιταχύνσεων λαμβάνεται σύμφωνα με τις παραγράφους 2.2.2.1, 2.2.2.2, 2.2.2.4, 2.2.2.6 και 2.2.2.7 του Ελληνικού Αντισεισμικού Κανονισμού (ΕΑΚ) και σύμφωνα με τις διατάξεις του παρόντος, αναφορικά με τους συντελεστές σπουδαιότητας γ_I, θεμελίωσης θ και μετελαστικής συμπεριφοράς q.

(2) Η κατακόρυφη συνιστώσα θα λαμβάνεται σύμφωνα με την παραγρ. 2.2.2.8 του ΕΑΚ (βλέπε § 2.3.2 του ΕΑΚ 2000)

(3) Ο Κύριος του Εργου έχει το δικαίωμα σε περιπτώσεις που με αιτιολογημένη απόφασή του ορίζει γέφυρες ως ιδιαίτερα σημαντικές, να προβεί σε σύνταξη ειδικής τεχνικής - σεισμολογικής μελέτης για τον καθορισμό των σεισμικών δράσεων σχεδιασμού, καθώς και να καθορίσει πρόσθετες ή διαφορετικές διατάξεις από τις αναφερόμενες στο παρόν κείμενο.

2.2 Συντελεστής Σπουδαιότητας γ_I

2.2.1 Γενικά

(1) Ανάλογα με τη σπουδαιότητα του έργου ο σεισμός σχεδιασμού μπορεί να ορισθεί με την επιλογή μιας αποδεκτής πιθανότητας υπέρβασης p, μέσα στην υπολογιστική διάρκεια ζωής t_d του έργου. Τότε η περίοδος ε-παναφοράς t_r του σεισμού σχεδιασμού προκύπτει από τη σχέση:

$$t_r = 1 / [1-(1-p)^{1/t_d}] \qquad (2.1)$$

(2) Σε γέφυρες αυτοκινητοδρόμων, εθνικών οδών και σιδηροδρόμων λαμβάνεται γενικά τιμή του συντελεστή σπουδαιότητας γ_1 = 1.0, που αντιστοιχεί σε περίοδο επαναφοράς του σεισμού σχεδιασμού περίπου 475 χρόνια. Ενας τέτοιος σεισμός έχει πιθανότητα υπέρβασης p κυμαινόμενη μεταξύ 0.10 και 0.19, για υπολογιστική διάρκεια ζωής του έργου t_d μεταξύ 50 και 100 χρόνων αντίστοιχα.

(3) Αν δεν γίνει αξιόπιστη στατιστική αξιολόγηση υπαρχόντων σεισμολογικών δεδομένων που να επιτρέπει, με συμφωνία του Κυρίου του Εργου, τον καθορισμό της σεισμικής δράσης με βάση τιμές των παραμέτρων σχεδιασμού (p και t_d ή t_r) διαφορετικές από τις προαναφερόμενες, η διαφοροποίηση του επιδιωκόμενου βαθμού ασφά-λειας μπορεί να επιτευχθεί μέσω των ακόλουθων τιμών του συντελεστή σπουδαιότητας γ_I:

Σχετικά προς τους χάλυβες είναι αναγκαία η εισαγωγή του συντελεστή ασφαλείας υλικού γ_m=1,15 για την εναρμόνιση προς τους Ευρωκώδικες.

Οι οριακές παραμορφώσεις σκυροδέματος και χάλυβα ελήφθησαν ίσες προς 3,5‰ και 5‰ αντίστοιχα, σύμφωνα προς DIN1045. Δεν θεωρήθηκε σκόπιμη η αύξηση του 5‰, δεδομένου ότι πρακτικά έχει ελάχιστη επιρροή στη διαστασιολόγηση της διατομής. Επισημαίνεται ότι οι παραμορφώσεις αυτές αναφέρονται μόνον στους ελέγχους διατομών και όχι σε ελέγχους πλαστιμότητας.

(2) Ο έλεγχος σε διάτμηση διαφοροποιείται για περιοχές εντός ή εκτός πλαστικής άρθρωσης. Η ουσιώδης διαφορά συνίσταται στη θεώρηση αποφλοιωμένης διατομής σε περιοχές πλαστικής άρθρωσης, σύμφωνα προς τον Ευρωκώδικα.

Το άνω όριο της τιμής τ_{03} (που ελέγχει σύμφωνα προς το DIN1045 την θλίψη της λοξής διαγωνίου) τίθεται ίσο προς $\beta_R/3\cdot1,50 \cong 0,225\beta_R$, όπου $\beta_R/3$ η οριακή διατμητική αντοχή και 1,50 ο συντελεστής ασφαλείας υλικού, ήτοι για B25
τ_{03} = 17,5/3·1,50= 3,93MPa έναντι 4,0 των Ευρωκωδίκων για C20/25.

Θεωρήθηκε επί πλέον σκόπιμη η επιβολή πλήρους διατμητικής κάλυψης σε διατομές πλαστικών αρθρώσεων, αφενός μεν διότι η συμμετοχή του σκυροδέματος στην ανάληψη τέμνουσας δεν είναι καθοριστική, αφ' ετέρου δε διότι η αποδιοργάνωση της διατομής (ιδιαίτερα σε υψηλές τιμές του δείκτη συμπεριφοράς) δεν την εξασφαλίζει.

Αντίθετα επετράπη, κυρίως για λόγους συμβατότητας προς το DIN1045, η μειωμένη διατμητική κάλυψη εκτός πλαστικών αρθρώσεων, χωρίς αύξηση όμως της τιμής τ_{02}.

(3) Επεβλήθη επίσης, σε αντιστοιχία προς τον Ευρωκώδικα, η ανάληψη της τέμνουσας σε αρμούς διακοπής, συνεκτιμώμενης όμως και της επιρροής του αξονικού φορτίου.

Σ.2.7.2 Ελεγχος εδάφους

(2) Η συνοπτική εκτίμηση της κατακόρυφης σεισμικής συνιστώσας βασίζεται στην παραδοχή ότι τα βάθρα ταλαντώνονται με μηδενική ιδιοπερίοδο ενώ τα οριζόντια στοιχεία σε κάποια ιδιοπερίοδο στην οροφή του φάσματος.

Σ.2.7.3 Εφέδρανα

(1) Ο έλεγχος των εφεδράνων τροποποιήθηκε μερικώς ως προς την Ε39/93, ταυτιζόμενος πλέον με τον έλεγχο του Ευρωκώδικα. Οι τροποποιήσεις αναφέρονται στα ελαστομεταλλικά εφέδρανα και είναι οι εξής:

α) ο συντελεστής συμπεριφοράς q λαμβάνεται γενικά ίσος προς 1,0 δεδομένου ότι το ελαστομεταλλικό εφέδρανο λειτουργεί β) Περιοχές εκτός πλαστικής άρθρωσης

Λαμβάνεται υπόψη στον έλεγχο η πλήρης διατομή του σκυροδέματος, ίση προς $b_w d$ σε ορθογωνικές διατομές ή $\pi d^2/4$ σε κυκλικές. Ο μοχλοβραχίων z για την εκτίμηση της διατμητικής τάσης τ_0 επιτρέπεται να λαμβάνεται ίσος προς 0,9d σε ορθογωνικές ή 0,75d σε κυκλικές διατομές.
Η μέγιστη διατμητική τάση τ_0 δεν επιτρέπεται να υπερβεί την τιμή $0{,}225\beta_R$.
Επιτρέπεται μειωμένη διατμητική κάλυψη κατά DIN 1045 εφόσον η διατμητική τάση τ_0 είναι μικρότερη του τ_{02}. Η μειωμένη τάση υπολογισμού του διατμητικού οπλισμού λαμβάνεται ίση προς τ_0^2/τ_{02}.

4) Πέραν των παραπάνω διατμητικών ελέγχων επιβάλλεται σε αρμούς διακοπής σκυροδέτησης η τήρηση της σχέσης

$$V \leq A_L \ \beta_s / 1{,}15 + \min N_{Ed} \qquad (2.12)$$

όπου

V η υπολογιστική τέμνουσα σύμφωνα προς την παρ. 2.6.4
A_L η διατομή του διαμήκους οπλισμού που διασχίζει τον αρμό
$\min N_{Ed}$ το ελάχιστο αξονικό φορτίο (θετικό όταν είναι θλιπτικό), λαμβανομένης υπόψη και της κατακόρυφης συνιστώσας του σεισμού.

2.7.2 Ελεγχος εδάφους

(1) Ο έλεγχος αντοχής εδάφους θα γίνεται σύμφωνα προς την παράγραφο 5.3.2 του ΕΑΚ. Όταν η φέρουσα ικανότητα πασσάλου υπολογίζεται σύμφωνα προς το DIN 4014, με βάση τον συνοπτικό και έμμεσο χαρακτηρισμό του εδάφους που προβλέπεται από τον κανονισμό αυτό, θα χρησιμοποιείται μειωτικός συντελεστής ασφαλείας v =1,30 στην τιμή της φέρουσας ικανότητας. Ο συντελεστής ασφαλείας v θα λαμβάνεται ίσος με 1,0, όταν η φέρουσα ικανότητα υπολογίζεται με βάση εδαφικές παραμέτρους σχεδιασμού (δηλ. με αντιπροσωπευτικές τιμές διαιρεμένες με μερικούς συντελεστές ασφαλείας).

(2) Στους ελέγχους αντοχής εδάφους, αν δεν γίνει ακριβέστερη εκτίμηση της συμβολής της κατακόρυφης συνιστώσας της σεισμικής δράσης, αυτή μπορεί να λαμβάνεται ίση προς $\pm\ 0{,}7 \cdot \alpha \cdot (G_\beta+2{,}5G_k)$ όπου G_β το βάρος του βάθρου και του θεμελίου (και των επικαθήμενων γαιών εφ' όσον υπάρχουν) και G_k η δράση του καταστρώματος.

(3) Σε φορείς ευαίσθητους σε διαφορικές καθιζήσεις θα ελέγχονται και οι επιπτώσεις τους πάνω στο φορέα. Σε κάθε περίπτωση δεν επιτρέπονται παραμένουσες υποχωρήσεις μεγαλύτερες των 40mm.

2.6 Δράσεις Ελέγχου

2.6.1 Υπολογιστική σεισμική ένταση

(1) Αν δεν γίνεται ακριβέστερη εκτίμηση του δυσμενέστερου συνδυασμού των διευθύνσεων του σεισμού ο έλεγχος επιτρέπεται να γίνεται για τη δυσμενέστερη από τις παρακάτω υπολογιστικές σεισμικές εντάσεις :

$$A_{Ex} \text{ “+” } 0.30A_{Ey} \text{ “+” } 0.30A_{Ez}$$
$$0.30A_{Ex} \text{ “+” } A_{Ey} \text{ “+” } 0.30A_{Ez}$$
$$0.30A_{Ex} \text{ “+” } 0.30\,A_{Ey} \text{ “+” } A_{ez} \qquad (2.8)$$

όπου : A_{Ex} είναι η τιμή οποιουδήποτε από τα εντατικά μεγέθη της διατομής (M_x, M_y, V_x, V_y, N), που προκύπτουν για σεισμό κατά τη διεύθυνση x.

και: A_{Ey} και A_{Ez} είναι η τιμή του ίδιου μεγέθους που προκύπτει για σεισμό κατά τη διεύθυνση y και z αντίστοιχα.

(“+” είναι το σύμβολο της επαλληλίας, εφόσον προκύπτει δυσμενέστερο αποτέλεσμα. Σημειώνεται ότι το πρόσημο των εντατικών μεγεθών εναλλάσσεται ανάλογα με τη φορά της σεισμικής δράσης).

2.6.2 Σεισμικός συνδυασμός δράσεων

(1) Ο σεισμικός συνδυασμός δράσεων ορίζεται ως εξής:

$$E_d = G_k \text{ “+” } P \text{ “+” } A_{Ed} \text{ “+” } \psi_{21}\ Q_{1k} \text{ “+” } Q_2 \qquad (2.9)$$

όπου:

G_k είναι το σύνολο των μονίμων δράσεων με τη χαρακτηριστική τους τιμή (ίδιο βάρος και πρόσθετα μόνιμα)

P είναι η τελική τιμή δράσεων από προένταση

A_{Ed} είναι ο δυσμενέστερος συνδυασμός δράσεων, όπως αυτές ορίσθηκαν στο παραπάνω εδάφιο 2.6.1(1).

Q_{1k} είναι η χαρακτηριστική τιμή του μεταβλητού φορτίου κυκλοφορίας (επιτρέπεται να λαμβάνεται ομοιόμορφα κατανεμημένο σε ολόκληρο το μήκος του φορέα)

ψ_{21} ο αντίστοιχος συντελεστής συνδυασμού ίσος προς 0,2 για οδικές γέφυρες και 0,3 για σιδηροδρομικές

Q_2 είναι η οιονεί μόνιμη τιμή δράσεων, με μεγάλη διάρκεια, (π.χ. ώθηση γαιών, άνωση, πίεση ροής κ.α.)

(2) Εντάσεις καταναγκασμού επιτρέπεται να μη λαμβάνονται υπόψη στο σεισμικό συνδυασμό δράσεων, με εξαίρεση την περίπτωση γεφυρών, στις οποίες η σεισμική δράση αναλαμβάνεται από ελαστομεταλλικά εφέδρανα. Στην περίπτωση αυτή τα αποτελέσματα που οφείλονται σε εντάσεις καταναγκασμού πρέπει να λαμβάνονται υπόψη.

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overturning moments, and other seismic force effects corresponding to these augmented story shears.

Any of a number of rational analyses could be used. Some published computer programs take P-delta effects into account. Computer programs used to account for P-delta effects must consider horizontal displacements which include inelastic (non-linear) action of the structural elements. In formula C1-1 the design story drift (elastic) Δ has been increased by the factor $3(R_w/8)$ to approximate the actual drift of the structure including inelastic action of the structural elements.

The columns of moment frames which are designed with P-delta effects included, need not have their bending stresses amplified as in AISC Formula 1.6-1a $(1 - f_a/F_e)$, or ACI Formula 10 - 8 (δ_s), since these factors were intended to account for P-delta effects.

Because the relative stiffness of lateral load resisting systems in higher seismic zones is required to be greater than those in lower seismic zones, it should be noted that P-delta effects for systems in lower seismic zones are potentially much more significant than for systems in higher seismic zones.

10. Vertical Component of Seismic Forces: The dead load of the members will usually assure against problems resulting from upward accelerations while the typical load factors will provide assurance against failure resulting from downward accelerations.

It was realized that, in general, beams in frames will not collapse as a result of vertical seismic loads since in the case of overstresses a catinary mode of load redistribution can carry the loads, thus preventing collapse. Since cantilevers do not have this continuity it was felt necessary to provide some additional assurance. Also, both simply supported and continuous prestressed beams should be checked for the reduced vertical load combination.

F. Dynamic Lateral Force Procedure

NEW SECTI

1. General: The dynamic analysis procedures described in this section are intended to incorporate the structure's dynamic analysis procedures, the scaling provisions contained in Section 1F5e differ from those for other structure types and soil conditions. In particular, these provisions specify that the design base shear and corresponding response parameters obtained from the dynamic analysis are not to be reduced if they exceed the values given in Section 1F5d(2). Therefore, for such conditions, the amplitude of the input horizontal ground motions must now be specified. Accordingly, this paragraph specifies that the input horizontal ground motion amplitudes for this case should correspond to a 10 percent probability of exceedance within a 50 year time period, as obtained from a properly justified probabilistic seismic hazard analysis for the site.

b. Vertical Ground Motion: Prior statistical analyses of ground motion records have shown that it is generally reasonable for design purposes to use a vertical component of ground motion whose peak amplitude is two-thirds of the peak amplitude of the horizontal motion. However, for unusual site conditions, a site-specific evaluation should be used to specify vertical ground motions for seismic design.

3. **Mathematical Model:** (to be provided)

4. **Description of Analysis Procedures:**

a. Response Spectrum Analysis: The dynamic analysis procedure described in this section uses a response spectrum representation of the seismic input motions. The procedure is applicable to linear elastic building models that are developed in accordance with the principles set forth in Section 1F3. It consists of the following steps [F4,F5]:

(1) Use principles of mechanics to compute the natural period and mode shape for the first N normal modes of the building model, where N is established in accordance with Section 1F5a.

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~~1.0 DL + 0.8 LL + 3(Rw/8)E~~
0.85 DL ± 3 (R_w/8) E

(1) The axial forces in such columns need not exceed the capability of other elements of the structure to transfer these loads to the column.

(2) Such columns shall be capable of carrying the above described axial forces without exceeding the axial strength of the column. For designs using working stress methods this strength may be determined using an allowable stress increase of 1.7.

(3) Such columns shall meet the following detailing requirements or member limitations:

(a) Section 3D for concrete in Seismic Zones 3 and 4 and Sections 4D, 4F4, and 4F7 for steel in Seismic Zones 2, 3 and 4.

(b) Section A.9.5. of ACI 318-83 for concrete in Seismic Zone 2.

(c) See Section 1J4 for overturning moments to be resisted at the foundation-soil interface.

8. **Story Drift Limitation.** Story drift is the displacement of one level relative to the level above or below due to the design lateral forces. Calculated drift shall include translational and torsional deflections.

a. Calculated story drift shall not exceed 0.04/R_w nor 0.005 times the story height for structures having a fundamental period of less than 0.7 seconds. For structures having a fundamental period of 0.7 seconds or greater the calculated story drift shall not exceed 0.03/R_w nor 0.004 times the story height. The period used in this determination shall be the same as that used for determining the base shear.

b. These drift limits may be exceeded where it is demonstrated that greater drift can be tolerated by both structural elements and nonstructural elements that could affect life safety.

c. The design lateral forces used to determine the calculated drift may be derived from a value of C resulting from a period determined from section 1E2b(2) method B neglecting the 80 percent limitation of Section 1E2b(2) and the lower bound limit of 0.075 for C/R_w.

9. **P-Delta Effects.** The resulting member forces and moments and the story drifts induced by P-delta effects shall be considered in the evaluation of overall structural frame stability. P-delta need not be considered where the story drift does not exceed 0.02/R_w times the story height.

10. **Vertical Component of Seismic Forces.** The following requirements apply in Seismic Zones 3 and 4 only.

a. Horizontal cantilever components shall be designed for a net upward force of ~~0.5 Z W_p~~.

b. In addition to all other applicable load combinations, horizontal prestressed components shall be designed using not more than 50 percent of the dead load for the gravity load, alone or in combination with the lateral force effects.

F. Dynamic Lateral Force Procedure.

1. **General.** Dynamic analyses procedures, where used, shall conform to the criteria established in this section. The analysis shall be based on ground motions defined using the procedures given in Section 1F2 and shall be performed using established principles of mechanics. Structures which are designed in accordance with this section shall comply with all other applicable requirements of these recommendations.

2. **Ground Motion.** The ground motion representation shall, as a minimum, be one having a ten percent probability of exceedance in 50 years and may be one of the following:

a. **Normalized Response Spectra.** The normalized response spectra given in Figure 1-B.

b. **Site Specific Design Spectra.** A site specific response spectrum shall be based on the geologic, tectonic, seismologic, and soil characteristics associated with the specific site. The spectra shall be developed for a damping ratio of 0.05 unless a different value is shown to be consistent with the anticipated structural behavior at the intensity of shaking established for the site.

c. **Time Histories.** Ground motion time histories developed for the specific site shall be representative of actual earthquake motions. Response spectra from time histories, either individually or in combination, shall approximate the site-specific design spectra conforming to paragraph b above.

d. **Structures on Soil Profile Type S_4.** The following requirements shall apply when required by Section 1D8b(4):

(1) The ground motion representation shall be developed in accordance with paragraphs b and c above and shall equal or exceed the motion having a 10 percent probability of exceedance in 50 years.

(2) Possible amplification of building response due to soil-structure interaction and lengthening of building period caused by inelastic behavior shall be considered.

(3) The base shear determined by these procedures may be reduced to a design base shear, V, by dividing by a factor not greater than the appropriate R_w factor for the structure but shall not be less than required by Section 1F5c(1).

e. **Vertical Component.** The vertical component of ground motion may be defined by scaling the corresponding adjusted horizontal accelerations by a factor of two-thirds. Alternative factors may be used when substantiated by site-specific data.

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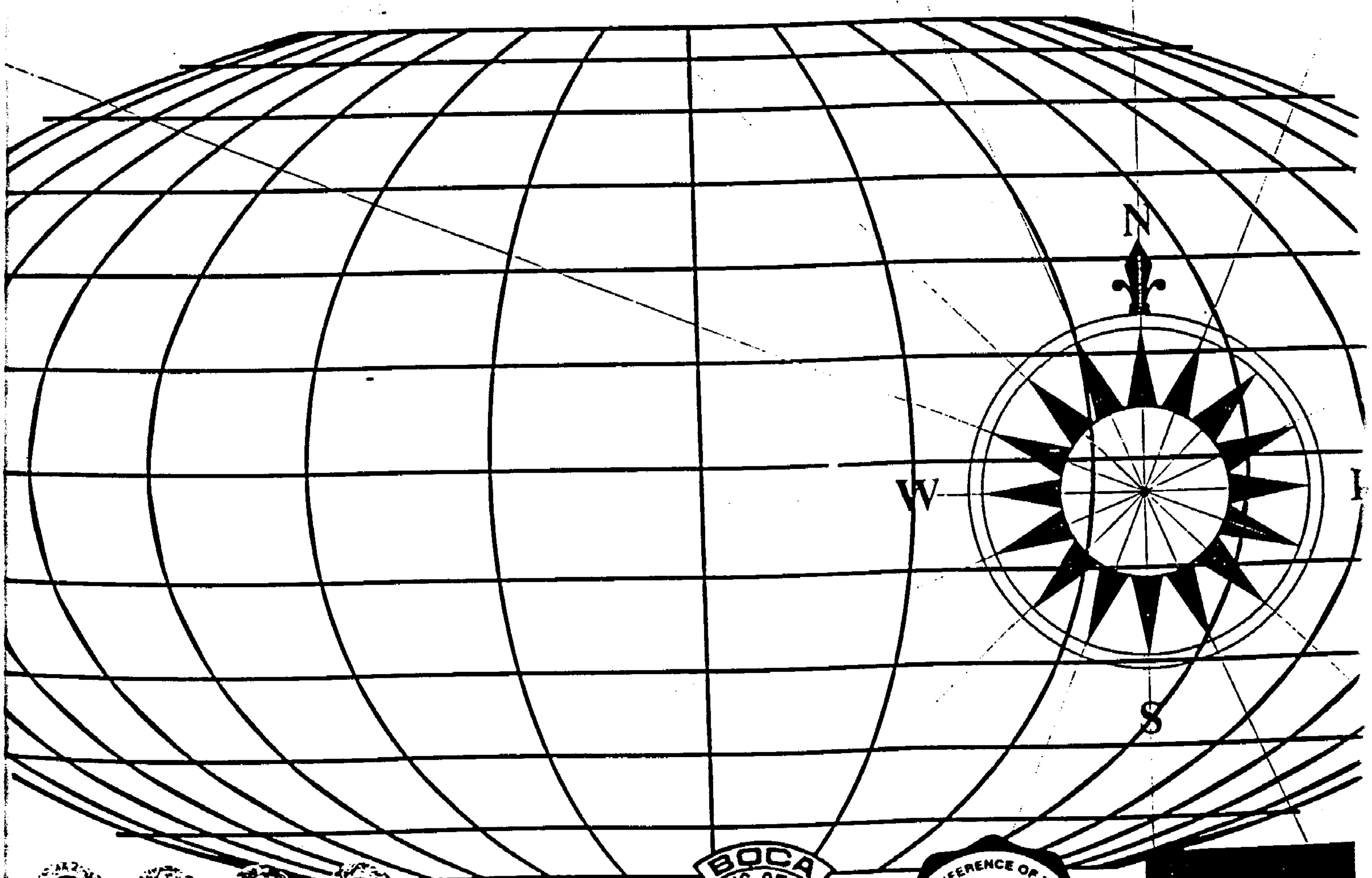

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In wood diaphragms, the continuous ties shall be in addition to the diaphragm sheathing. Anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension. The diaphragm sheathing shall not be considered effective as providing the ties or struts required by this section.

In metal deck diaphragms, the metal deck shall not be used as the continuous ties required by this section in the direction perpendicular to the deck span.

Diaphragm to wall anchorage using embedded straps shall be attached to or hooked around the reinforcing steel or otherwise terminated so as to directly transfer force to the reinforcing steel.

1620.2.2 Direction of seismic load. For structures that have plan structural irregularity Type 5 in Table 1616.5.1, the critical direction requirement of Section 1620.1.10 shall be deemed satisfied if components and their foundations are designed for the following orthogonal combination of prescribed loads.

One hundred percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction. The combination requiring the maximum component strength shall be used. Alternatively, the effects of the two orthogonal directions are permitted to be combined on a square root of the sum of the squares (SRSS) basis. When the square root of the sum of the squares method of combining directional effects is used, each term computed shall be assigned the sign that will result in the most conservative result.

1620.3 Seismic Design Category D. Structures assigned to Seismic Design Category D shall conform to the requirements of Section 1620.2 for Seismic Design Category C and to the following.

1620.3.1 Plan or vertical irregularities. For buildings having a plan structural irregularity of Type 1a, 1b, 2, 3 or 4 in Table 1616.5.1 or a vertical structural irregularity of Type 4 in Table 1616.5.2, the design forces determined from Section 1617.4.1 shall be increased 25 percent for connections of diaphragms to vertical elements and to collectors, and for connections of collectors to the vertical elements.

1620.3.2 Vertical seismic forces. Horizontal cantilever and horizontal prestressed components shall be designed to resist the vertical component of earthquake ground motion. This requirement is considered to be met if:

1. The load combinations used in designing such components include E as defined in Equation 16-29, and
2. Such components are designed to resist, in addition to the applicable load combinations of Section 1605, a minimum net upward force of 0.2 times the dead load.

1620.3.3 Diaphragms. The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached elements to maintain structural integrity under the individual loading and continue to support the prescribed loads.

Floor and roof diaphragms shall be designed to resist design seismic forces determined in accordance with Equation 16-65 as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad \text{(Equation 16-65)}$$

where:

F_i = The design force applied to Level i.

F_{px} = The diaphragm design force.

w_i = The weight tributary to Level i.

w_{px} = The weight tributary to the diaphragm at Level x.

The force determined from Equation 16-65 need not exceed $0.3\ S_{DS} I_E w_{px}$ but shall not be less than $0.15\ S_{DS} I_E w_{px}$, where S_{DS} is the design spectral response acceleration at short period determined in Section 1615.1.3 and I_E is the occupancy importance factor determined in Section 1616.2. When the diaphragm is required to transfer design seismic force from the vertical-resisting elements above the diaphragm to other vertical-resisting elements below the diaphragm due to offsets in the placement of the elements or to changes in relative lateral stiffness in the vertical elements, these forces shall be added to those determined from Equation 16-65 and to the upper and lower limits on that formula.

1620.3.4 Collector elements. Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing resistance to those forces.

Collector elements, splices and their connections to resisting elements shall resist the forces determined in accordance with Equation 16-65. In addition, collector elements, splices and their connections to resisting elements shall have the design strength to resist the earthquake loads as defined in the Special Load Combinations of Section 1605.4.

Exception: In structures, or portions thereof, braced entirely by light frame shear walls, collector elements, splices and their connections to resisting elements

TABLE 1616.5.2
VERTICAL STRUCTURAL IRREGULARITIES

	IRREGULARITY TYPE AND DESCRIPTION	REFERENCE SECTION	SEISMIC DESIGN CATEGORY[a] APPLICATION
1a	Stiffness Irregularity—Soft Story A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.	1616.6.3 Table 1616.6.3	D, E and F D, E and F
1b	Stiffness Irregularity—Extreme Soft Story An extreme soft story is one in which the lateral stiffness is less than 60 percent of that in the story above or less than 70 percent of the average stiffness of the three stories above.	1620.4.1 1616.6.3 Table 1616.6.3	E and F D, E and F D, E and F
2	Weight (Mass) Irregularity Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 1616.6.3	D, E and F
3	Vertical Geometric Irregularity Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any story is more than 130 percent of that in an adjacent story.	Table 1616.6.3	D, E and F
4	In-plane Discontinuity in Vertical Lateral-Force-Resisting Elements An in-plane offset of the lateral-force-resisting elements greater than the length of those elements or a reduction in stiffness of the resisting element in the story below.	1620.3.1 1616.6.3 1620.1.9	D, E and F D, E and F B, C, D, E and F
5	Discontinuity in Capacity—Weak Story A weak story is one in which the story lateral strength is less than 80 percent of that in the story above. The story strength is the total strength of seismic-resisting elements sharing the story shear for the direction under consideration.	1620.1.3 1616.6.3 1620.4.1	B, C, D, E and F D, E and F E and F

a. Seismic Design Category is determined in accordance with Section 1616.

1616.6.2 Seismic Design Categories B and C. Except as permitted by Section 1616.6, the analysis procedures in Section 1617.4 shall be used for structures assigned to Seismic Design Category B or C (Section 1616) or a more rigorous analysis is permitted to be made.

1616.6.3 Seismic Design Categories D, E and F. The analysis procedures identified in Table 1616.6.3 shall be used for structures assigned to Seismic Design Category D, E or F (see Section 1616), or a more rigorous analysis shall be made. For regular structures five stories or fewer in height having a period T, as determined in Section 1617.4.2, of 0.5 seconds or less, the design spectral response accelerations, S_{DS} and S_{D1}, need not exceed the values calculated using values of S_S and S_1, respectively, of 1.5g and 0.6g.

For the purposes of this section, structures shall be considered regular if they do not have plan irregularities 1a, 1b or 4 of Table 1616.5.1 or vertical irregularities 1a, 1b, 4 or 5 of Table 1616.5.2.

SECTION 1617
EARTHQUAKE LOADS—MINIMUM DESIGN LATERAL FORCE AND RELATED EFFECTS

1617.1 Seismic load effect E and E_m. Seismic load effect, E and E_m, for use in the load combinations of Section 1605 shall be determined as follows.

1617.1.1 Seismic load effect E. Where the effects of gravity and the seismic ground motion are additive, seismic load, E, for use in Formulas 16-5, 16-10, and 16-17 shall be defined by Equation 16-28:

$$E = \rho Q_E + 0.2 S_{DS} D \qquad \textbf{(Equation 16-28)}$$

where:

D = The effect of dead load.

E = The combined effect of horizontal and vertical earthquake-induced forces.

ρ = A reliability factor based on system redundancy obtained in accordance with Section 1617.2.

TABLE 1616.6.3
ANALYSIS PROCEDURES FOR SEISMIC DESIGN CATEGORIES D, E OR F

STRUCTURE DESCRIPTION	MINIMUM ALLOWABLE ANALYSIS PROCEDURE FOR SEISMIC DESIGN
1. Seismic Use Group I buildings of light-framed construction three stories or less in height and of other construction, two stories or less in height with flexible diaphragms at every level.	Simplified procedure of Section 1617.5.
2. Regular structures, other than those in Item 1 above, up to 240 feet in height.	Equivalent lateral-force procedure (Section 1617.4).
3. Structures that have vertical irregularities of Type 1a, 1b, 2 or 3 in Table 1616.5.2, or plan irregularities of Type 1a or 1b of Table 1616.5.1, and have a height exceeding five stories or 65 feet and structures exceeding 240 feet in height.	Modal analysis procedure (Section 1618).
4. Other structures designated as having plan or vertical irregularities.	Equivalent lateral-force procedure (Section 1617.4) with dynamic characteristics included in the analytical model.
5. Structures with all of the following characteristics: - located in an area with S_{D1} of 0.2 or greater, as determined in Section 1615.1.3; - located in an area assigned to Site Class E or F, in accordance with Section 1615.1.1 and; - with a natural period T of 0.7 second or greater, as determined in Section 1617.4.2.	Modal analysis procedure (Section 1618). A site- specific response spectrum shall be used but the design base shear shall not be less than that determined from Section 1617.4.1.

For SI: 1 foot = 304.8 mm.

Q_E = The effect of horizontal seismic forces.

S_{DS} = The design spectral response acceleration at short periods obtained from Section 1615.1.3 or 1615.2.5.

Where the effects of gravity and seismic ground motion counteract, the seismic load, E, for use in Formulas 16-6, 16-12 and 16-18 shall be defined by Equation 16-29.

$$E = \rho Q_E - 0.2 S_{DS} D \qquad \textbf{(Equation 16-29)}$$

Design shall use the load combinations prescribed in Section 1605.2 for strength or load and resistance factor design methodologies, or Section 1605.3 for allowable stress design methods.

1617.1.2 Maximum seismic load effect, E_m. The maximum seismic load effect, E_m, shall be used in the special seismic load combinations in Section 1605.4.

Where the effects of the seismic ground motion and gravity loads are additive, seismic load, E_m, for use in Formula 16-19 shall be defined by Equation 16-30.

$$E_m = \Omega_0 Q_E + 0.2 S_{DS} D \qquad \textbf{(Equation 16-30)}$$

Where the effects of the seismic ground and gravity loads counteract, seismic load, E_m, for use in Formula 16-20 shall be defined by Equation 16-31.

$$E_m = \Omega_0 Q_E - 0.2 S_{DS} D \qquad \textbf{(Equation 16-31)}$$

where E, Q_E, S_{DS} are as defined above and Ω_0 is the system overstrength factor as given in Table 1617.6.

The term $\Omega_0 Q_E$ need not exceed the maximum force that can be transferred to the element by the other elements of the lateral-force-resisting system.

Where allowable stress design methodologies are used with the special load combinations of Section 1605.4, design strengths are permitted to be determined using an allowable stress increase of 1.7 and a resistance factor, ϕ, of 1.0. This increase shall not be combined with increases in allowable stresses or load combination reductions otherwise permitted by this code or the material reference standard except that combination with the duration of load increases permitted in Chapter 23 is permitted.

1617.2 Redundancy. A redundancy coefficient, ρ, shall be assigned to all structures in accordance with this section, based on the extent of structural redundancy inherent in the lateral-force-resisting system.

1617.2.1 Seismic Design Category A, B or C. For structures assigned to Seismic Design Category A, B or C (see Section 1616), the value of the redundancy coefficient ρ is 1.0.

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establish the dependence of the response spectrum on the thickness and v_s-value of the soft clay/silt layer and on the stiffness contrast between this layer and the underlying materials.

(5) The value of the damping correction factor η can be determined by the expression

$$\eta = \sqrt{10/(5+\xi)} \geq 0,55 \qquad (3.5)\ \{31\}$$

where

ξ viscous damping ratio of the structure, expressed in percent.

(6) If for special studies a viscous damping ratio different from 5% is to be used, this value will be given in the relevant Parts of Eurocode 8.

(7) The elastic displacement response spectrum, $DS_e(T)$, shall be obtained by direct transformation of the elastic acceleration spectrum, $S_e(T)$, using the following expression:

$$DS_e(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 \qquad (3.6)$$

(8) Expression (3.6) shall normally be applied for vibration periods not exceeding 3,0 seconds. For structures with vibration periods greater than 3,0 seconds, a more complete definition of the Type 1 elastic spectrum is presented in Annex A in terms of displacement response spectrum. {32}

3.2.2.3 Vertical elastic spectrum {33}

(1) P The vertical component of the seismic action should be represented by a response spectrum, $S_{ve}(T)$, derived using expressions (3.7)-(3.11) in combination with the values of the control parameters presented in tables 3.2 and 3.3.

$0 \leq T \leq T_B$:

$$S_{ve}(T) = a_{vg} \cdot \left[1 + \frac{T}{T_B} \cdot (\eta_v \cdot 3,0 - 1)\right] \qquad (3.7)$$

$T_B \leq T \leq T_C$:

$$S_{ve}(T) = a_{vg} \cdot \eta_v \cdot 3,0 \qquad (3.8)$$

$T_C \leq T \leq T_D$:

$$S_{ve}(T) = a_{vg} \cdot \eta_v \cdot 3,0\left[\frac{T_C}{T}\right] \qquad (3.9)$$

$4\sec \geq T \geq T_D$:

$$S_{ve}(T) = a_{vg} \cdot \eta_v \cdot 3{,}0\left[\frac{T_C . T_D}{T^2}\right] \quad (3.10)$$

$$\eta_v = \sqrt{2{,}72/(0{,}72+\xi)} \quad (3.11)$$

Table 3.4: Values of parameters describing the vertical elastic response spectrum

Spectrum	a_{vg}/a_g	T_B	T_C	T_D
Type 1	0,90	0,05	0,15	1,0
Type 2	0,45	0,05	0,15	1,0

(2) It should be noted that the ordinates of the vertical response spectrum are independent of the subsoil class. However, the values in table 3.4 and expressions (3.7)-(3.11) are only applicable for subsoil classes A, B, C, D and E, and not for special classes S_1 and S_2.

3.2.2.4 Peak ground displacement

(1) Unless special studies based on the available information indicate, otherwise the value d_g of the peak ground displacement may be estimated by means of the following expression:

$$d_g = [0{,}025] \cdot a_g \cdot S \cdot T_C \cdot T_D \quad (3.12)\ \{34\}$$

with the values of a_g, S, T_c, T_D as defined in 3.2.2.2.

3.2.2.5 Design spectrum for elastic analysis

(1) The capacity of structural systems to resist seismic actions in the non-linear range generally permits their design for forces smaller than chose corresponding to a linear elastic response.

(2) To avoid explicit inelastic structural analysis in design, the energy dissipation capacity of the structure, through mainly ductile behaviour of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one, henceforth called "design spectrum", This reduction is accomplished by introducing the behaviour factor q.

(3) The behaviour factor q is an approximation of the ratio of the seismic forces, that the structure would experience if its response was completely elastic with 5% viscous damping, to the minimum seismic forces that may be used in design - with a conventional elastic response model - still ensuring a satisfactory response of the structure. The values of the behaviour factor q, which also accounts for the influence of

the viscous damping being different from 5%, are given for the various materials and structural systems and according to various ductility levels in the relevant Parts of Eurocode 8.

(4) P For the reference return period the design spectrum, $S_d(T)$, is defined by the following expressions:

$$0 \le T \le T_B : S_d(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot \left(\frac{2,5}{q} - 1\right)\right] \qquad (3.13)$$

$$T_B \le T \le T_C : S_d(T) = a_g \cdot S \cdot \frac{2,5}{q} \qquad (3.14)$$

$$T_C \le T \le T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \dfrac{2,5}{q} \cdot \left[\dfrac{T_C}{T}\right] \\ \ge [0,20] \cdot a_g \end{cases} \qquad (3.15)$$

$$T_D \le T : \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \dfrac{2,5}{q} \cdot \left[\dfrac{T_C T_D}{T^2}\right] \\ \ge [0,20] \cdot a_g \end{cases} \qquad (3.16)$$

where

$S_d(T)$ ordinate of the design spectrum, which is normalised by g,

q behaviour factor.

(5) Values of the parameters S, T_B, T_C, and T_D are given in tables 3.2, 3.3 and 3.4. {36}

(6) P The design spectrum as defined above is not sufficient for the design of structures with base-isolation or energy-dissipation-systems.

3.2.3 Alternative representations of the seismic action

3.2.3.1 Time - history representation

3.2.3.1.1 General

(1) P The seismic motion may also be represented in terms of ground acceleration time-histories and related quantities (velocity and displacement).

(2) P When a spatial model is required, the seismic motion shall consist of three simultaneously acting accelerograms. The same accelerogram may not be used simultaneously along both horizontal directions. Simplifications are possible according to the relevant Parts of Eurocode 8.

(3) For the five subsoil classes A, B, C, D and E the values of the parameters S, T_B, T_C and T_D are given in table 3.2. for Type 1 Spectrum and table 3.3 for Type 2 Spectrum, as defined in Section 3.2.2.1 {25}.

Note:

For special site-classification studies referred in 3.1.1 (4) National Authorities should provide the corresponding changes of parameter S.

Table 3.2: Values of the parameters describing the Type 1 elastic response spectrum {26}

Subsoil Class	S	T_B	T_C	T_D
A	1,0	0,15	0,4	2,0
B	1,1	0,15	0,5	2,0
C	1,35	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Table 3.3: Values of the parameters describing the Type 2 elastic response spectrum {26}

Subsoil Class	S	T_B	T_C	T_D
A	1,0	0,05	0,25	1,2
B	1,2	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

(3) For sites with ground conditions matching the classes S_1 and S_2 special studies for the definition of the seismic action may be required. {29}

(4) Special attention should be paid in the case of a deposit of sub-soil S_1 {30}. Such soils typically have very low values of v_s, low internal damping and an abnormally extended range of linear behaviour and can therefore produce anomalous seismic site amplification and soil-structure interaction effects; see Section 6 of Part 5. In this case, a special study for the definition of the seismic action should be carried out, in order to

The maximum value of each action effect on the structure due to the two horizontal components of the seismic action may then be estimated by the square root of the sum of the squared responses to each horizontal component.

(3) As an alternative to paragraph (2) the action effects due to the combination of the horizontal components of the seismic action may be computed using the two following combinations:

a) E_{Edx} "+" 0,30 E_{Edy} (4.20)

b) 0,30 E_{Edx} "+" E_{Edy} (4.21)

where

"+" implies "to be combined with",

E_{Edx} action effects due to the application of the seismic action along the chosen horizontal axis x of the structure,

E_{Edy} action effects due to the application of the same seismic action along the orthogonal horizontal axis y of the structure.

(4) The sign of each component in the above combinations shall be taken as the most unfavourable for the effect under consideration.

(5) P When using nonlinear static (pushover) analysis and applying a spatial model, the combination rules of (2), (3) above should be applied, considering as E_{dx} the forces and deformations due to the target displacement in the X direction and as E_{dy} the forces and deformations due to the target displacement in the Y direction. The internal forces resulting from the combination shall not exceed the corresponding capacities.

(6) P When using nonlinear time-history analysis and employing a spatial model of the structure, simultaneously acting accelerograms shall be considered for both horizontal components.

(7) P For buildings satisfying the regularity criteria in plan and in which walls are the only horizontal load resisting components, the seismic action may be assumed to act separately and without combinations (2) and (3) above, along the two main orthogonal horizontal axes of the structure.

4.4.3.5.2 Vertical component of the seismic action

(1) P The vertical component of the seismic action, as defined in clause 4.2.1.(3) of Part 1-1. shall be taken into account in the following cases:

- Horizontal or nearly horizontal structural members spanning 20 meters or more;
- Horizontal or nearly horizontal cantilever components;
- Horizontal or nearly horizontal prestressed components;
- Beams supporting columns.

(2) In general, the analysis for determining the effects of the vertical component of the seismic action can be made based on a partial model of the structure which includes the elements under consideration and takes into account the stiffness of the adjacent elements.

(3) The effects of the vertical component need only be considered for the elements under consideration and their directly associated supporting elements or substructures.

(4) In case the horizontal components of the seismic action are also relevant for these elements, the following three combinations may be used for the computation of the action effects:

a) $0,30\,E_{Edx}$ "+" $0,30\,E_{Edy}$ "+" E_{Edz} (4.22)

b) E_{Edx} "+" $0,30\,E_{Edy}$ "+" $0,30\,E_{Edz}$ (4.23)

c) $0,30\,E_{Edx}$ "+" E_{Edy} "+" $0,30\,E_{Edz}$ (4.24)

where

E_{Edx} and E_{Edy} see 4.4.3.5.1 (3),

E_{Edz} action effects due to the application of the vertical component of the design seismic action as defined in 3.2.2.3 Σημείωση $q = 1.0$

(5) If nonlinear static (pushover) analysis is performed, the vertical component of the seismic action may be neglected.

4.4.4 Displacement analysis

(1) P If linear analysis is performed the displacements induced by the design seismic action shall be calculated on the basis of the elastic deformation of the structural system by means of the following simplified expression:

$$d_s = q_d\, d_e \tag{4.25}$$

where

d_s displacement of a point of the structural system induced by the design seismic action.

q_d displacement behaviour factor, assumed equal to q unless otherwise specified in Section 5 to 9,

d_e displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum according to 3.2.2.5.

The value of d_s does not need to be larger than the value derived from the elastic spectrum

(2) P When determining the displacements d_e, the torsional effects of the seismic action shall be taken into account.

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CALTRANS

SEISMIC DESIGN CRITERIA

VERSION 1.1

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Method 2 The application of the ground motion along the principal axes of individual components. The ground motion must be applied at a sufficient number of angles to capture the maximum deformation of all critical components.

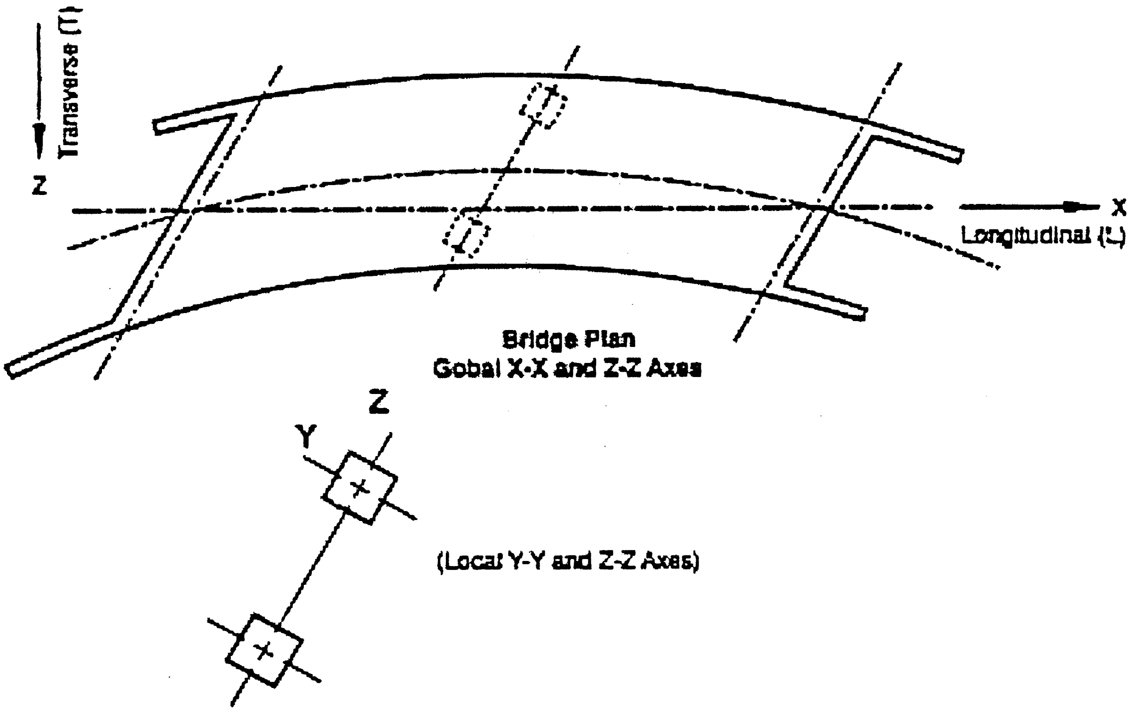

Figure 2.1 Local–Global Axis Definition

2.1.3 Vertical Ground Motions

A vertical acceleration response spectra analysis is not required for Ordinary Standard bridges. For bridge sites where the peak rock acceleration exceeds 0.5g, an equivalent static vertical load shall be applied to the superstructure to estimate the effects of vertical acceleration[2]. The superstructure shall be designed to resist the applied vertical force as specified in Section 7.2.2. A case-by-case determination on the effect of vertical load is required for Non-standard and Important bridges.

2.1.4 Vertical/Horizontal Load Combination

A combined vertical/horizontal load analysis is not required for Ordinary Standard bridges.

[2]This is an interim method of approximating the effects of vertical acceleration on superstructure capacity. The intent is to ensure all superstructure types, especially lightly reinforced sections such as P/S box girders, have a nominal amount of mild reinforcement available to resist the combined effects of dead load, earthquake, and prestressing in the upward or downward direction. This is a subject of continued study.

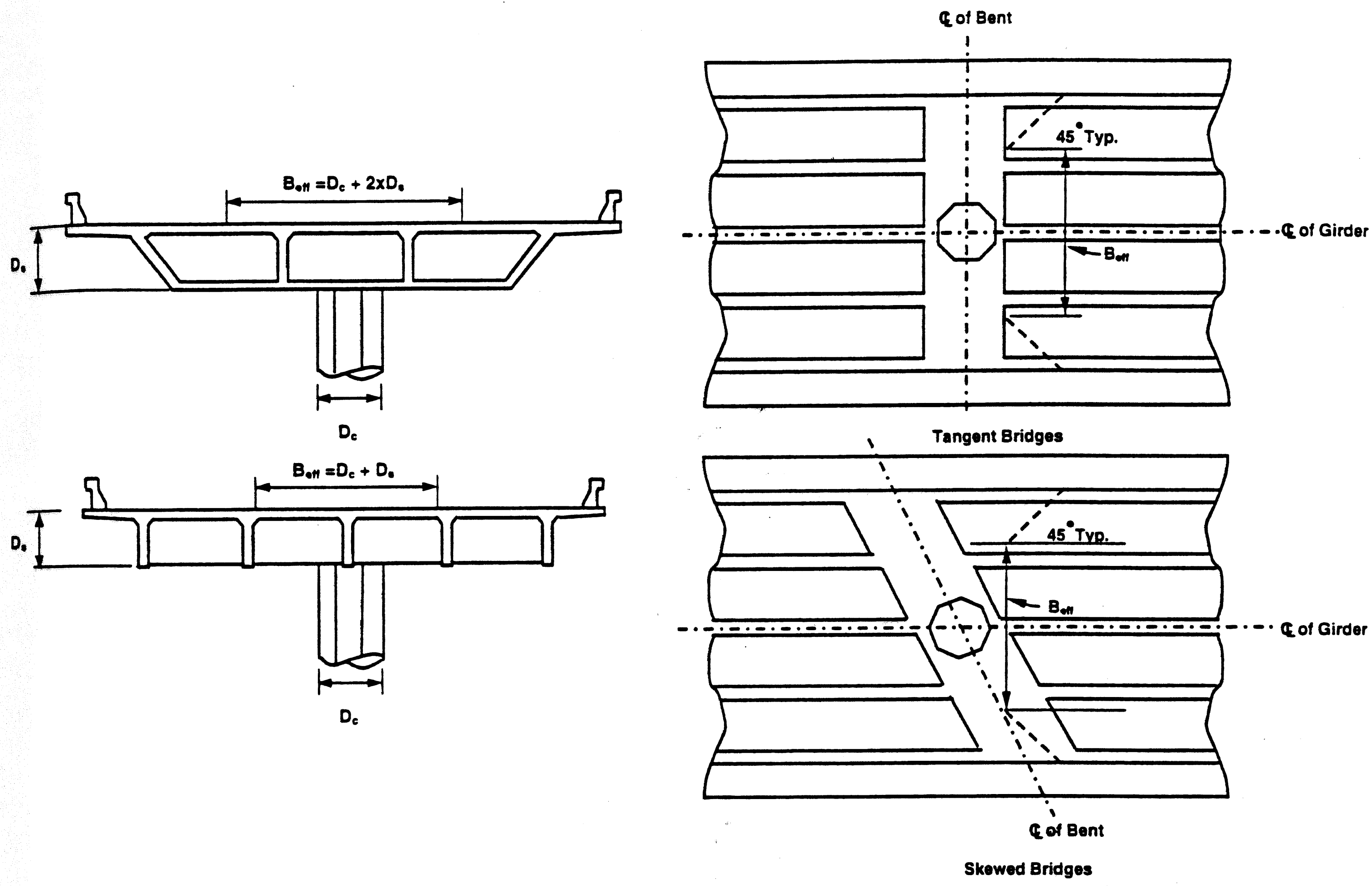

Figure 7.2 Effective Superstructure Width

7.2.2 Vertical Acceleration

If vertical acceleration is considered, per Section 2.1, a separate analysis of the superstructure's nominal capacity shall be performed based on a uniformly applied vertical force equal to 25% of the dead load applied upward and downward, see Figure 7.3. The superstructure flexural capacity shall be based only on continuous mild reinforcement distributed evenly between the top and bottom slabs. The effects of dead load, primary and secondary prestressing shall be ignored. The continuous steel shall be spliced with "service level" couplers as defined in Section 8.1.3, and may be integrated with the mild reinforcement required for other load cases. Splicing of the vertical acceleration steel in critical zones such as mid-span or near the supports should be avoided.

The longitudinal side reinforcement in the girders shall be capable of resisting 125% of the dead load shear at the bent face by means of shear friction. The enhanced side reinforcement shall extend continuously 25 ft (7 m) beyond the face of the bent cap.

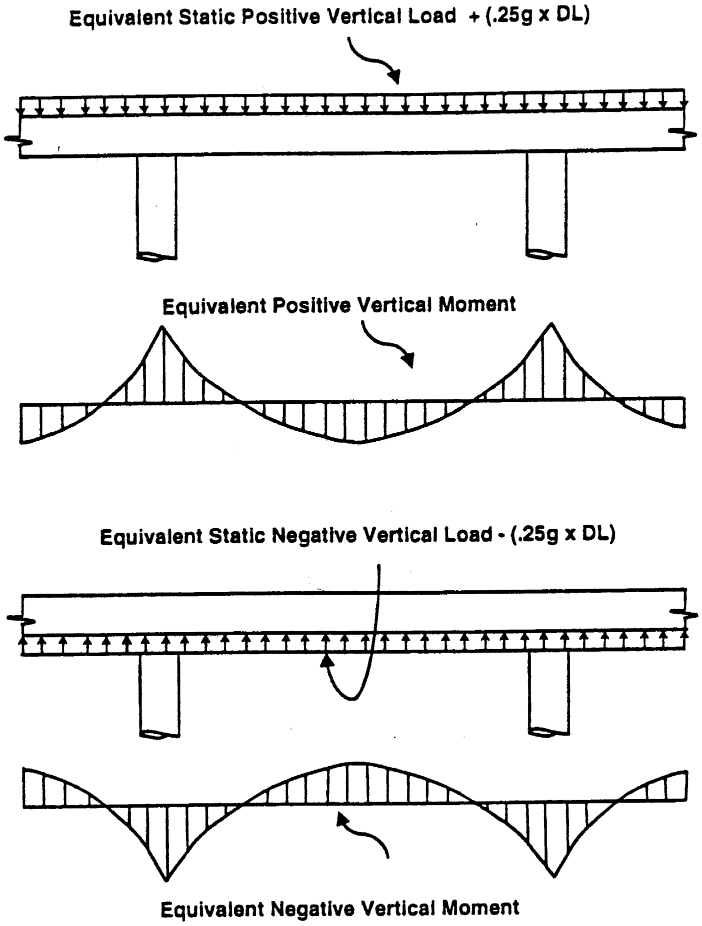

Figure 7.3 Equivalent Static Vertical Loads & Moments

7.2.3 Pre-cast Girders

Pre-cast girders shall be designed to remain essentially elastic when resisting the column overstrength moments and shears. Recent research has confirmed the viability of pre-cast spliced girders with integral column/superstructure details that effectively resist longitudinal seismic loads. This type of system is considered non-standard until design details and procedures are formally adopted. In the interim, project specific design criteria shall be developed per MTD 20-11.

7.2.4 Slab Bridges

Slab bridges shall be designed to meet all the strength and ductility requirements as specified in the SDC.

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Effect of Vertical Ground Motions on the Structural Response of Highway Bridges

by

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SECTION 1
INTRODUCTION

The objective of this study is to determine under what conditions the vertical component of seismic ground motion is critical in determining the demands placed on key elements of typical highway structures. In current design practice, the vertical component of motion is not usually included in the analysis of bridges or buildings, though the Uniform Building Code [1997] does specify increased multipliers on dead loads that are intended to approximate its effects. These multipliers are 0.9DL and 1.2DL for non-isolated buildings, and 0.8DL and 1.2DL for isolated buildings. Vertical spectral shapes are not defined in current bridge design codes, however when the vertical component is included, it is normally specified as a spectrum with an amplitude two-thirds of the horizontal spectrum. In recent years, various researchers have conducted statistical studies on large numbers of strong ground motion records that show this vertical-to-horizontal ratio grossly underestimates the strength of the vertical component in the near fault (< 5km) region and at short periods.

The research approach in the current work is to analyze a representative group of bridges with a range of input ground motions that include and exclude the vertical component of motion. The results of the dynamic analyses are compared for both cases and conclusions are drawn as to when the vertical component can be safely ignored and when its effects should be included in the design of highway bridges. The scope of the study involves linear analyses of finite element models of six typical highway bridges using a broad range of input motions. These elastic models were obtained from the Berger/ABAM series of bridges assembled as seismic design examples for the Federal Highway Administration. Both time history and response spectrum analyses are performed, and results compared. One bridge from this group that shows sensitivity to vertical excitation is selected for nonlinear dynamic analyses, again including and excluding the vertical component of motion.

On the basis of the results from these linear and nonlinear analyses, recommendations are made regarding cases where vertical motions should explicitly be included in design, where the effects of vertical motions can be adequately addressed by simple load combination rules, and finally, cases where the impact of vertical motions is less than 10% and thus can be ignored from a design perspective.

Section 2 gives a summary of previous research work done on the effects of the vertical component of motion on bridge decks, piers, foundations, bearings and hinges. Section 3 provides a description of each of the six bridges analyzed, including their physical dimensions, element properties used in the structural model, and vertical dynamic characteristics. Section 4 describes the characteristics of the vertical component of motion and gives details of response spectra and frequency scaled time history records used for input motions in the bridge analyses. Details of the parameters used in the linear dynamic analysis of each bridge are given in Section 5. Section 6 presents the results of the analyses and provides recommendations based on results from these analyses. Results from the response spectrum and time history analyses are compared. The effects of varying vertical deck stiffness and foundation fixity on the vertical structural response are reported. Response spectrum analysis results using three different directional combination rules are compared.

This report ends with five appendices, which contain response ratios computed from response spectrum analyses of the six bridges, response ratios computed from response spectrum analyses for varying deck stiffnesses and foundation fixity in bridge numbers 4 and 6, mode shapes with modal mass participation ratios greater than 10% for the six bridges, SAP2000 input files, and the ANSR-II input file for Bridge 6. These appendices are provided on MCEER's web site at *http://mceer.buffalo.edu*.

8.2.1 Conclusions related to the ground motions

1. The vertical component of seismic ground motion at close-in soil sites and distant rock and soil sites is relatively rich in short period waves that arrive earlier than the largest horizontal motions.
2. Records at some close-in rock sites (less than 10 to 15 km) exhibit longer period motions in the vertical component that have similar arrival times and more similar frequencies to the largest horizontal motions.
3. At both rock and soil sites, and for magnitudes above 5.5 and distances less than 40 km, for periods in the range of approximately 0.2 to 3.0 seconds the vertical to horizontal (V/H) spectral ratio (as computed by Silva [1997]) is less than the commonly used value of 2/3. For periods shorter than approximately 0.2 seconds, the V/H spectral ratio is greater than 2/3.
4. For soil sites with distances up to approximately 20 km, in the period range of approximately 0.02 to 0.15 seconds the V/H ratio exceeds 1.0 for all magnitudes above 5.5.
5. For rock sites with distances up to approximately 10 km, in the period range of approximately 0.03 to 0.1 seconds the V/H ratio exceeds 1.0 for magnitudes above 6.5.
6. The V/H spectral ratio for magnitude 6.5 events has a peak value of about 1.1 and 1.9 for rock and soil respectively, and the peak ratio increases to about 1.3 (rock) and 2.6 (soil) for magnitude 7.5 events. The V/H spectral ratio increases with increasing earthquake magnitude for all periods less than approximately 1.0 seconds. At longer periods, the magnitude dependence is much less apparent.

8.2.2 Structural Response of Bridges in the Linear Range

Most of the design office analyses that are performed on bridges are based on linear elastic models using the response spectrum method of analysis. Very rarely is the vertical component included in such analyses. If vertical motions are included in an analysis, they generally use two-thirds of the amplitude of the horizontal response spectra. Bridge codes to date have not provided load multipliers or specific vertical response spectra that allow for the impact of vertical motions.

All six bridges included in this study have been analyzed using linear elastic models with and without the vertical component of motion. Both response spectra and time history analyses have been performed. The conclusions are as follows:

1. Bridges with the greatest percentage of modal mass lying in the range of the peak spectral acceleration of the vertical response spectra experience the greatest impact from the vertical seismic motions. An attempt was made to assess the amount of modal mass less than 0.2 seconds that caused significant vertical response. Unfortunately, the six bridges used in this study were not sufficient to develop a specific recommendation on this issue.
2. Tables 6.8 to 6.15, Figures 6.8 to 6.23, and Tables 8.1 and 8.2 give DL multipliers that may be applied to various response quantities in order to eliminate the need to include the vertical component of motion in a dynamic analysis. Three different response ratios (3/2, 3/DL, (3-2)/DL) were examined in this study. It was found that the (3-2)/DL ratio gave the best practical measure of the impact of the vertical component of motion on bridges. The (3-2)/DL ratio is computed by dividing the difference in absolute response values from the three-component input and two-component input loading cases by the dead load only response value. Tables 8.1 and 8.2 present the ratios for magnitude 6.5 and 7.5 events, respectively, for both rock and soil conditions. These ratios increase substantially as the bridge site gets closer to the fault.
3. In order to envelop the design forces as a function of DL on all bridges for both magnitude 6.5 and 7.5 events, the multipliers get quite large, especially when the bridge site is within 10km of a fault. For magnitude 6.5 events a DL multiplier of ±0.4DL would envelop all forces in the 20-50km range. A multiplier of ±0.7DL would be required in the 0-20km range except for the mid-

span moment in which a ±1.4DL multiplier would be necessary. For magnitude 7.5 events, a DL multiplier of ±0.6DL would envelop all forces in the 20-60km range. A multiplier of ±1.0DL would be required in the 0-20km range except for the mid-span moment in which a ±1.9DL multiplier would be necessary. As a consequence, it would seem prudent to consider the use of an appropriate DL multiplier on all bridge deck design forces and column axial design forces when the bridge location is 20 to 50km from a fault. When the bridge site has a fault distance of less than 20 km, it would seem prudent to require the inclusion of a vertical ground motion analysis in the analysis of a bridge rather than specifying very large multipliers. Beyond 60 km from the fault, the value of ±10% of the dead load design value would adequately account for the impact of the vertical component of motion on all vertical design forces. As a consequence, the impact of the vertical component of motion could be ignored when a bridge site is greater than 60km from a fault.

4. Values of horizontal response quantities are not significantly affected by the vertical component of motion.
5. Results from linear response spectrum analyses using the CQC modal combination method and the SRSS directional combination method are mostly within 10% of the average linear response from time history analyses using five records frequency-scaled to the input spectra.
6. Response values from a modal analysis using vertical spectra computed from attenuation relationships by Abrahamson and Silva [1997], and Sadigh et al [1993; 1997] can be up to 40% greater or less than those obtained from vertical spectra that have a spectral amplitude equal to 2/3 of the horizontal spectra. It be should be noted that the "2/3 spectra" generally give conservative results for vertical deck response quantities; but for pier axial force, the results are mostly unconservative. For this reason, it is recommended that the use of the 2/3 multiplier to obtain the vertical spectra from the horizontal spectra should be discontinued.
7. Softening of a bridge deck due to cracking during an earthquake will generally reduce the effect the vertical component of motion has on the bridge (see Tables 6.19 and 6.20). As a consequence, the DL multipliers shown in Tables 6.8 to 6.15, Tables 8.1 and 8.2 and Figures 6.8 to 6.23 are conservative in that no deck stiffness reduction is included in their development.
8. Bridge models with fixed foundations give higher absolute response values for a three component input whereas flexible foundations tend to give higher (3-2)/DL ratios (see Tables 6.23 and 6.24).
9. Vertical shear at mid-span may need to be checked in bridges that are located within 10km of a fault and are designed for M7.5 loading, have uneven span lengths, and have columns that are effectively fixed to the deck.
10. The early arrival of the vertical component of motion does not have a significant effect on the structural response of typical highway bridges.
11. A magnitude 7.5 event and soil site conditions produces the highest (3-2)/DL ratios for pier axial force for all distances, and for deck shear at the pier and moment at mid-span at distances beyond 10 km. Rock site conditions produce the highest ratios for these two quantities for distances less than 10 km and for deck moment over the pier for all distances.
12. A comparison of results from modal analyses using the following three directional combination rules (a) SRSS rule (b) 100% + 30% rule (c) 100% + 40% rule showed that using the SRSS method produced results that were closest to the average result from time history analyses using five spectrum compatible records (see Tables 6.25 and 6.26).

8.2.3 Structural Response of Bridges in the Nonlinear Range

The results presented in Section 8.2.2 are in conflict with current design practice since it has been assumed to date that the vertical ground motion does not have a significant impact on the design of a bridge. One of the obvious questions resulting from the linear analyses is what impact does the nonlinear response of key components of the bridge have on the results discussed in Section 8.2.2. Deck softening was discussed in Item 6 of Section 8.2.2 although this was not part of a nonlinear study. Unfortunately, it was not possible to perform an extensive study on the nonlinear response of each bridge. Bridge 6 was re-analyzed incorporating the nonlinear response of the piers. These nonlinear results were compared with the linear response results and the following observations resulted from the limited nonlinear modeling of this one bridge.

1. Including nonlinear behavior in the piers strongly influences the horizontal response of the structure although the horizontal displacements are not significantly impacted.
2. Response values for horizontal quantities are not significantly affected by the vertical component of motion in the bridge studied. However, earlier research has indicated some sensitivity of the horizontal response to the inclusion of vertical motions in inelastic bridges
3. (3-2)/DL ratios are slightly greater but essentially the same for the nonlinear and linear response of the majority of response quantities for this one bridge.

Generalizations from the above observations are not warranted until further nonlinear analysis are performed on a wider range of bridge structures.

8.3 Recommendations

The results of this study are important and will be a surprise to many because it has been commonly assumed to date that vertical ground motions do not have a significant impact on the response of a bridge. As a consequence, current bridge design codes do not incorporate any design provisions to account for the response resulting from vertical ground motions. This clearly is not a valid assumption for bridge sites located within 20km of a fault and for some bridges, the vertical response may be important when the site is located within 40km of a fault.

There are two methods that design codes can utilize to address the vertical response issue. The first is simply to require the inclusion of a vertical component of motion in the design and analysis process when a bridge site is within some distance of a fault (e.g. 10km). The second is more complex from a code perspective but more straightforward from a design perspective. It involves the incorporation of a percentage (e.g. ±40%) of the dead load design force on the design of the deck, columns and bearings without the necessity of performing a vertical analysis. The complexity of this method arises because the design forces that need to be addressed vary significantly by the distance of the bridge site from the fault and, as the site gets within 10km of a fault, some of the percentages get into the 70 – 190% range. It would therefore seem prudent under these circumstances to require the inclusion of the vertical component of motion rather than have very high multipliers that of necessity have to envelop the results obtained in this study.

In order to aid in the development of future code provisions, the following recommendations are offered for consideration.

1. The values for DL multipliers in Tables 8.1 to 8.2 should be considered for inclusion in code provisions in lieu of conducting explicit vertical analyses. If the design process is to be simplified as

much as possible, an envelop of the multipliers could be considered and consideration should be given to having appropriate multipliers when the bridge site is located within certain distances, i.e., 0 to 20km; 20 to 40km; 40 to 60km. At a distance greater than 60km the impact of the vertical response is less than ± 10% of the dead load design value for all of the design quantities included in this study and can therefore be ignored. Decisions will need to be made on the distance from a fault and whether or not to envelop the magnitude 7.5 and 6.5 events or to have separate multipliers for different magnitude events.

2. Vertical motions should be explicitly included in the analysis and design of most bridges within 10km of a major fault. This will avoid the use of very high envelop multipliers, e.g. ± 1.9 on the DL design forces.
3. If linear analysis is appropriate for a particular bridge, response spectrum analyses can accurately represent the vertical response of complex three-dimensional bridges to multi-component seismic excitation.
4. Use of a vertical spectrum equal to 2/3 of the corresponding horizontal spectrum is not recommended.
5. Additional nonlinear work is required to validate the conclusions reached in this research, although it does appear that further nonlinear analyses will reduce the impact of the vertical ground motions.

Table 8.1 Fault Distance Zones and Corresponding Dead Load Multiplier for ALL BRIDGES Observed for Rock and Soil Site Conditions and a Magnitude 6.5 Event

<table>
<tr><th rowspan="3">Response Quantity</th><th colspan="9">Fault Distance Zones (km)</th></tr>
<tr><th>0-5</th><th>5-10</th><th>10-15</th><th>15-20</th><th>20-25</th><th>25-30</th><th>30-35</th><th>35-40</th><th rowspan="2">>40 or given value</th></tr>
<tr><th colspan="2">0-10</th><th colspan="2">10-20</th><th colspan="2">20-30</th><th colspan="2">30-40</th></tr>
<tr><td rowspan="3">Pier Axial Force DL Multiplier</td><td colspan="9"></td></tr>
<tr><td>0.7</td><td>0.5</td><td>0.3</td><td>0.3</td><td>0.2</td><td></td><td>0.1</td><td></td><td rowspan="2">0.1</td></tr>
<tr><td colspan="2">0.7</td><td colspan="2">0.3</td><td colspan="2"></td><td colspan="2"></td></tr>
<tr><td rowspan="3">Deck Shear Force at Pier DL Multiplier</td><td colspan="9"></td></tr>
<tr><td>0.7</td><td>0.5</td><td>0.4</td><td>0.3</td><td>0.2</td><td></td><td></td><td>0.1</td><td rowspan="2">0.1</td></tr>
<tr><td colspan="2">0.7</td><td colspan="2">0.4</td><td colspan="2"></td><td colspan="2"></td></tr>
<tr><td rowspan="3">Deck Bending Moment at Pier DL Multiplier</td><td colspan="9"></td></tr>
<tr><td>0.6</td><td>0.4</td><td>0.3</td><td>0.3</td><td>0.2</td><td></td><td></td><td>0.1</td><td rowspan="2">0.1</td></tr>
<tr><td colspan="2">0.6</td><td colspan="2">0.3</td><td colspan="2"></td><td colspan="2"></td></tr>
<tr><td rowspan="3">Deck Shear Force at Mid-Span DL Multiplier</td><td colspan="9"></td></tr>
<tr><td>0.1</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="2">0.1</td></tr>
<tr><td colspan="2"></td><td colspan="2"></td><td colspan="2"></td><td colspan="2"></td></tr>
<tr><td rowspan="3">Deck Bending Moment at Mid-Span* DL Multiplier</td><td colspan="9"></td></tr>
<tr><td>1.4</td><td>1.0</td><td>0.7</td><td>0.5</td><td>0.4</td><td>0.4</td><td>0.3</td><td>0.3</td><td rowspan="2">0.1 (>50)</td></tr>
<tr><td colspan="2">1.4</td><td colspan="2">0.7</td><td colspan="2">0.4</td><td colspan="2">0.3</td></tr>
</table>

Footnotes

(1) The DL Multiplier values given above are in addition to the dead load; thus, an actual "load factor" would be **1.0 plus/minus** *the above numbers.*

*(2) *Broekhuizen[1997](see Section 2, 2.1 Decks) concluded that prestressed spans will not experience significant damage for upward accelerations of up to 1g applied to the superstructure.*

(3) The Live Load (LL) typically used in the design of bridge types shown in this study is in the range of 20-30% of the Dead Load (DL).

Table 8.2 Fault Distance Zones and Corresponding Dead Load Multiplier for ALL BRIDGES Observed for Rock and Soil Site Conditions and a Magnitude 7.5 Event

Response Quantity	Fault Distance Zones (km)								
	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	>40 or given value
	0-10		10-20		20-30		30-40		
Pier Axial Force DL Multiplier	0.9	0.6	0.4	0.3	0.2				0.1
	0.9		0.4						
Deck Shear Force at Pier DL Multiplier	1.0	0.7	0.5	0.4	0.3	0.3	0.2		0.1 (>50)
	1.0		0.5		0.3				
Deck Bending Moment at Pier DL Multiplier	1.0	0.7	0.5	0.4	0.3	0.3	0.2		0.1 (>50)
	1.0		0.5		0.3				
Deck Shear Force at Mid-Span DL Multiplier			0.1						0.1
	See Section 6.5								
Deck Bending Moment at Mid-Span* DL Multiplier	1.9	1.4	1.0	0.8	0.6	0.6	0.5	0.4	0.1 (>60)
	1.9		1.0		0.6		0.5		

Footnotes

(1) The DL Multiplier values given above are in addition to the dead load; thus, an actual "load factor" would be **1.0 plus/minus** *the above numbers.*

*(2) *Broekhuizen[1997](see Section 2, 2.1 Decks) concluded that prestressed spans will not experience significant damage for upward accelerations of up to 1g applied to the superstructure.*

(3) The Live Load (LL) typically used in the design of bridge types shown in this study is in the range of 20-30% of the Dead Load (DL).

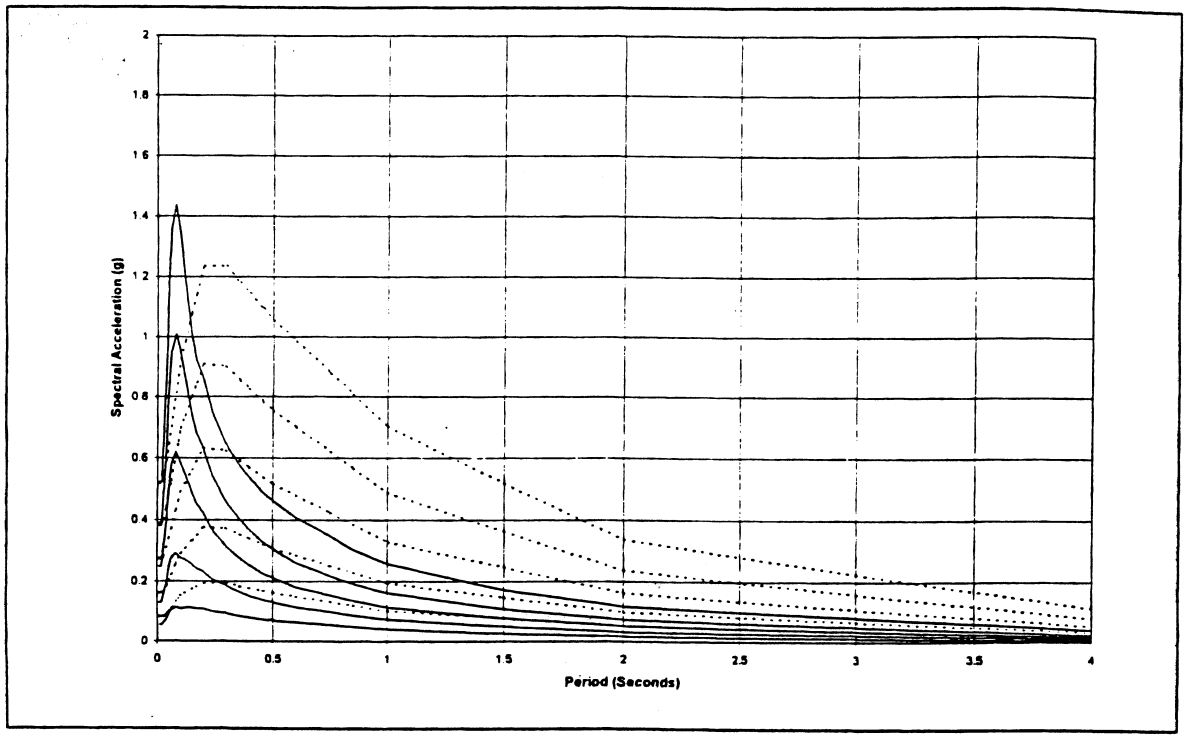

Figure 4.7 Horizontal (dotted lines) and Vertical Spectra for Fault Distances 1, 5, 10, 20, 40 km; Magnitude 6.5 and Soil Site Conditions

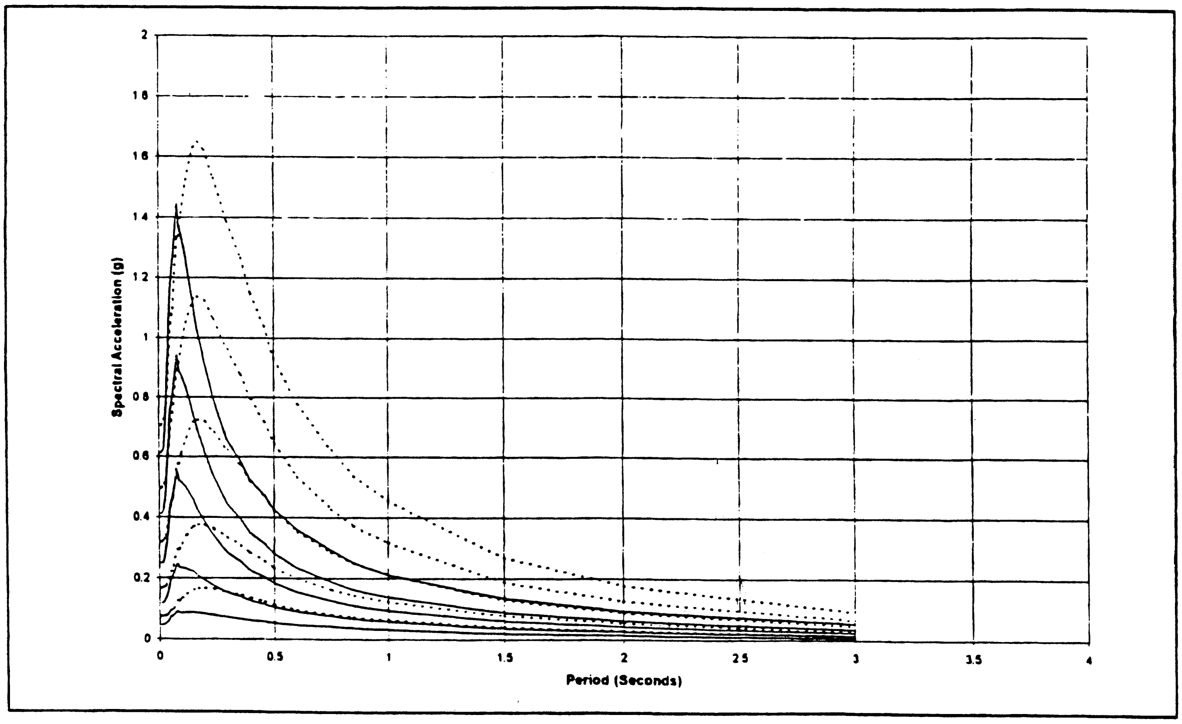

Figure 4.8 Horizontal (dotted lines) and Vertical Spectra for Fault Distances 1, 5, 10, 20 40 km; Magnitude 6.5 and Rock Site Conditions

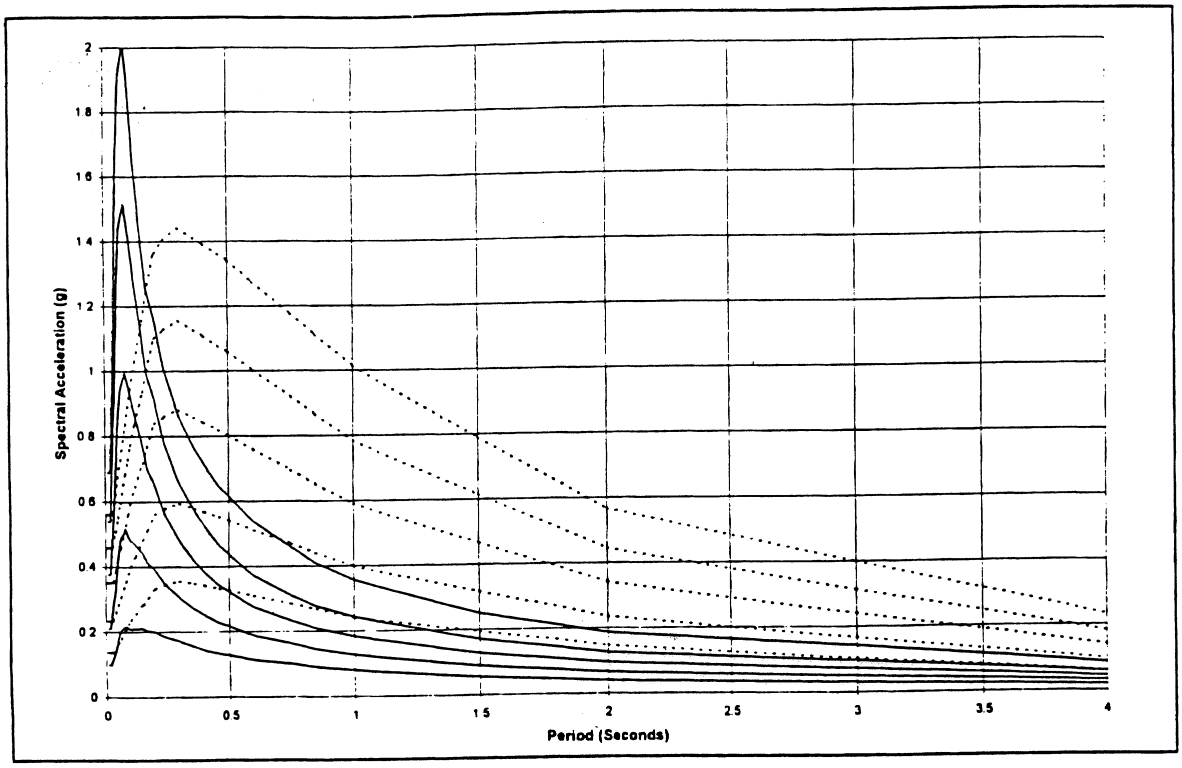

FIGURE 4.9 Horizontal (dotted lines) and Vertical Spectra for Fault Distances 1, 5, 10, 20, 40 km; Magnitude 7.5 and Soil Site Conditions

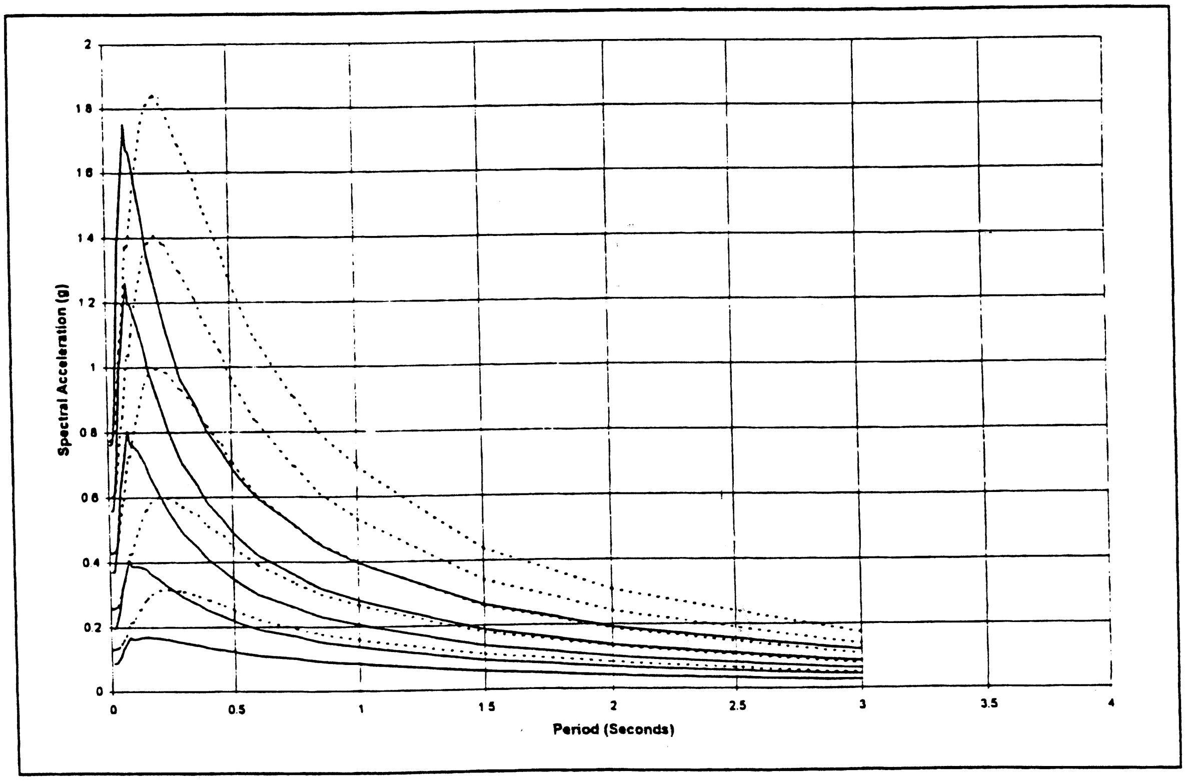

FIGURE 4.10 Horizontal (dotted lines) and Vertical Spectra for Fault Distances 1, 5, 10, 20, 40 km; Magnitude 7.5 and Rock Site Conditions

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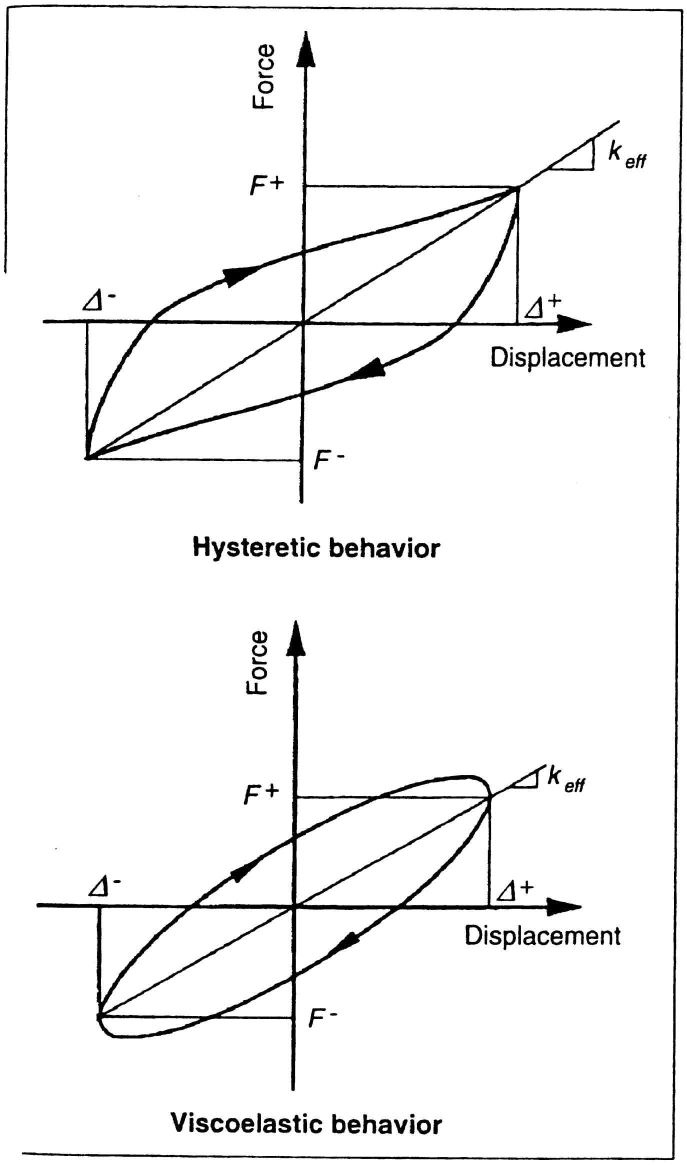

gure C9-11 Definition of Effective Stiffness of Seismic Isolation Devices

behavior of a group of bearings is small and can be neglected. Al-Hussaini et al. (1994) provided experimental results that demonstrate this behavior up to the point of imminent bearing uplift. Similar results are likely for elastomeric bearings.

The effect of vertical ground acceleration is to modify the load on the isolators. If it is assumed that the building is rigid in the vertical direction, and axial forces due to overturning moments are absent, the axial loads can vary between $W(1 - \ddot{U}/g)$ and $W(1 + \ddot{U}/g)$, where $\ddot{U}$ is the peak vertical ground acceleration. However, recognizing that horizontal and vertical ground motion components are likely not correlated unless in the near field, it is appropriate to use a combination rule that uses only a fraction of the peak vertical ground acceleration. Based on the use of 50% of the peak vertical ground acceleration, maximum and minimum axial loads on a given isolator may be defined as:

$$N_c = W(1 \pm 0.20 S_{DS}) \qquad \text{(C9-19)}$$

where the plus sign gives the maximum value and the minus sign gives the minimum value. Equation C9-19 is based on the assumption that the short-period spectral response parameter, S_{DS}, is 2.5 times the peak value of the vertical ground acceleration. For analysis for the Maximum Considered Earthquake, the axial load should be determined from

$$N_c = W(1 \pm 0.20 S_{MS}) \qquad \text{(C9-20)}$$

Equations C9-19 and C9-20 should be used with caution if the building is located in the near field of a major active fault. In this instance, expert advice should be sought regarding correlation of horizontal and vertical ground motion components.

Load N_c represents a constant load on isolators, which can be used for determining the effective stiffness and area of the hysteresis loop. To obtain these properties, the characteristic strength Q (see Figure C9-11) is needed. For sliding isolators, Q can be taken as equal to $f_{max}N_c$, where f_{max} is determined at the bearing pressure corresponding to load N_c. For example, for a sliding bearing with spherical sliding surface of radius R_o (see Figure C9-8), the effective stiffness and area of the loop at the design displacement D are:

$$k_{eff} = \left(\frac{1}{R_o} + \frac{f_{max}}{D}\right) N_c \qquad \text{(C9-21)}$$

$$\text{Loop Area} = 4 f_{max} N_c D \qquad \text{(C9-22)}$$

C. Nonlinear Models

For dynamic nonlinear time-history analysis, the seismic isolation elements should be explicitly