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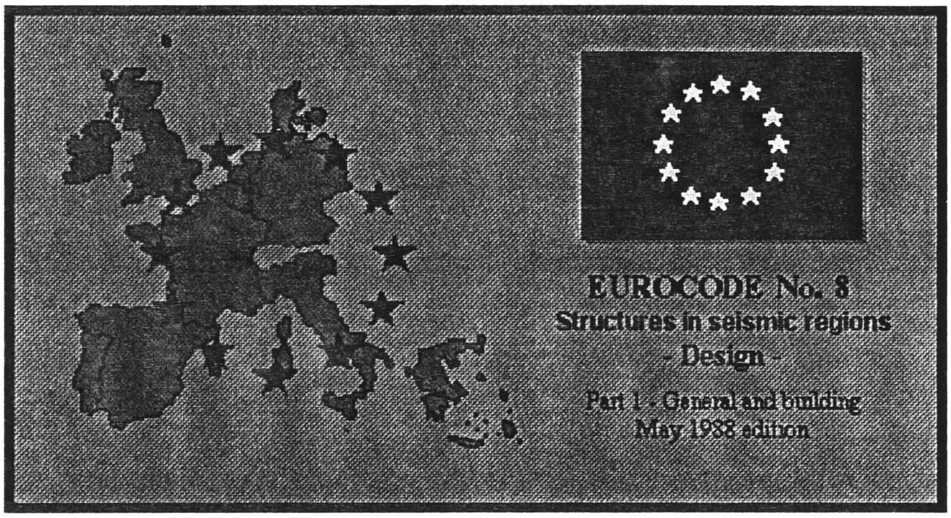
ΕΡΓΑΣΤΗΡΙΟ ΣΤΑΤΙΚΗΣ & ΑΝΤΙΣΕΙΣΜΙΚΩΝ ΕΡΕΥΝΩΝ



ΕΥΡΩΚΩΔΙΚΑΣ No. 8
ΜΕ ΤΗ ΜΟΡΦΗ ΕΜΠΕΙΡΟΥ ΣΥΣΤΗΜΑΤΟΣ

Β. ΚΟΥΜΟΥΣΗΣ, Χ. ΓΑΝΤΕΣ, Π. ΓΕΩΡΓΙΟΥ, ΧΡ. ΔΗΜΟΥ

ΤΕΛΙΚΗ ΕΚΘΕΣΗ ΕΣΑΕ-93

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ΠΡΟΛΟΓΟΣ

Η εξέλιξη των ηλεκτρονικών υπολογιστών καθώς και η ανάπτυξη προγραμμάτων ανάλυσης και σχεδιασμού έργων Πολιτικού Μηχανικού επιβάλλουν και την ανάπτυξη των κανονισμών έργων Πολιτικού Μηχανικού σε μορφή προγραμμάτων Η/Υ.

Η συνήθης πρακτική, που ακολουθείται κατά την ανάπτυξη λογισμικού, ενσωματώνει στη ροή του προγράμματος ανάλυσης και τους απαραίτητους ελέγχους αντοχής με τη μορφή αλγοριθμικών προγραμμάτων, απ' όπου διέρχονται ομάδες αποτελεσμάτων. Η λύση αυτή τείνει σήμερα να αντικατασταθεί από την ενεργοποίηση μόνον των διατάξεων που χρειάζονται σε κάθε μεμονωμένη περίπτωση, γεγονός που απαιτεί την οργάνωση των δεδομένων με τη μορφή βάσεων δεδομένων.

Η αντιμετώπιση αυτή αφορά κυρίως τα αλγοριθμικά τμήματα ενός κανονισμού, δηλ. αυτά που κωδικοποιούνται σε βήματα υπολογισμών. Ενας κανονισμός όμως περιέχει και πολλές διατάξεις με τη μορφή συστάσεων, δηλ. διατάξεων με ποιοτικό χαρακτήρα, που δεν ποσοτικοποιούνται κατ' ανάγκη, όπως π.χ διατάξεις για τη μόρφωση του φέροντα οργανισμού, την εξιδανίκευση του προσομοιώματος κλπ. Είναι λοιπόν σημαντικό, το σύνολο των διατάξεων ενός κανονισμού να παρέχεται σε ηλεκτρονική μορφή και προς διάφορες κατευθύνσεις.

Η μία κατεύθυνση είναι προς το μηχανικό, που θέλει να κατανοήσει τις διατάξεις ενός κανονισμού, και να έχει μία γρήγορη αναφορά, προσπέλαση και διασταύρωση διατάξεων κατά τη φάση μόρφωσης του φέροντα οργανισμού της κατασκευής αλλά και κατά τον έλεγχο και τη διερεύνηση των ενδιάμεσων λύσεων πριν την οριστικοποίηση της μελέτης ενός έργου.

Η άλλη κατεύθυνση είναι η σύνδεση του πληροφοριακού συστήματος με τα προγράμματα ανάλυσης και διαστασιολόγησης παρέχοντας αυτόματα αφενός τους αλγορίθμους και αφετέρου τα ποιοτικά χαρακτηριστικά των επιλεγόμενων λύσεων, συνολικά για τη μόρφωση του φορέα αλλά και σε επιμέρους θέματα, όπως συμμόρφωση με ομάδες διατάξεων του κανονισμού κλπ.

Η παρούσα εργασία αφορά την ανάπτυξη ενός συστήματος της πρώτης κατηγορίας. Απευθύνεται προς το Μηχανικό που επιθυμεί να ανατρέξει στις διατάξεις του Ευρωκώδικα Νο. 8 και να παρακολουθήσει τη λογική των διατάξεων του, καθώς επίσης και τη μεταξύ τους διασύνδεση που αφορά επιμέρους ενότητες ή και ολόκληρο το κανονισμό.

Το σύστημα αυτό της παρούσας εργασίας αναπτύχθηκε με τη μορφή ενός έμπειρου συστήματος που διατρέχει τη λογική του κανονισμού και ενός συστήματος hypertext που παρέχει εξηγήσεις κατά έναν ιεραρχημένο τρόπο.

Το έμπειρο σύστημα αναπτύχθηκε με βάση τη λογική της γλώσσας Prolog. Η αναπαράσταση της γνώσης έγινε με κανόνες της μορφής IF...THEN...ELSE. Ο συμπερασματικός μηχανισμός που χρησιμοποιήθηκε είναι της μορφής οπισθοδρομικού συμπερασματικού μηχανισμού (backward chaining inference engine), ενώ τα προς απόδειξη κατηγορήματα αφορούν, κατά ιεραρχίες, όλα τα επιμέρους θέματα του κανονισμού όπως εμφανίζονται στα περιεχόμενα του. Η ανάπτυξη του συστήματος έγινε χρησιμοποιώντας το περιβάλλον ανάπτυξης έμπειρων συστημάτων Knowledge Pro [12]. Το σύστημα αυτό λειτουργεί στο περιβάλλον των MS-Windows 3.x [13] προσφέροντας τα πλεονεκτήματα του αντικειμενοστρεφούς προγραμματισμού (object oriented programming).

Η ανάπτυξη του συστήματος Hypertext έγινε σε περιβάλλον MS-Windows 3.x χρησιμοποιώντας τις ρουτίνες του SDK (Software Development Kit) των MS-Windows 3.x.

Για την ανάπτυξη του όλου συστήματος απαιτήθηκε η ηλεκτρονική αναπαραγωγή ολόκληρου του κώδικα σε ενιαία μορφή.

Η ερευνητική ομάδα που μετείχε στην ανάπτυξη του έργου, διάρκειας ενός έτους, απαρτιζόταν από τους Β. Κουμούση Επικ. Καθ. ΕΜΠ, επιστημονικό υπεύθυνο, Χ. Γαντέ Λέκτορα ΕΜΠ, Π. Γεωργίου Πολ. Μηχανικό, Μεταπτυχιακό Υπότροφο ΕΜΠ, και τον Χ. Δήμου Πολ. Μηχανικό, Μεταπτυχιακό Σπουδαστή ΕΜΠ καθώς και ο Καθηγητής του Πανεπιστημίου του Heriot-Watt, B.H.V. Topping ο οποίος μετείχε ώς σύμβουλος.

ΠΕΡΙΕΧΟΜΕΝΑ

ΦΙΛΟΣΟΦΙΑ ΚΑΝΟΝΙΣΜΩΝ ΣΥΝΟΛΙΚΗΣ ΑΝΤΟΧΗΣ

Η σύγχρονες αντιλήψεις για τη σύνταξη των κανονισμών ακολουθούν σήμερα την ενιαία λογική της μεθόδου συνολικής αντοχής που στηρίζεται σε ορισμένες αρχές σχεδιασμού.

Αντίθετα προς τη λογική των κανονισμών επιτρεπόμενων τάσεων, όπου η κατασκευή θεωρείται ότι λειτουργεί στην ελαστική περιοχή, στους κανονισμούς συνολικής αντοχής οι κατασκευές επιτρέπεται και είναι επιθυμητό να εισέρχονται και στην πλαστική περιοχή για ορισμένες ακραίες φορτίσεις που έχουν μικρή πιθανότητα εμφάνισης κατά τη διάρκεια ζωής του έργου. Ο σχεδιασμός όμως, αντί να διερευνά όλες τις ενδιάμεσες καταστάσεις, επικεντρώνει την προσοχή του σε ορισμένες διακριτές καταστάσεις τις ονομαζόμενες οριακές καταστάσεις (limit states). Οι οριακές καταστάσεις κατατάσσονται σε δύο ομάδες, στις οριακές καταστάσεις αστοχίας και στις οριακές καταστάσεις λειτουργικότητας. Οι πρώτες αναφέρονται στην κατάσταση πριν την κατάρρευση της κατασκευής, ενώ οι άλλες στην ελαστική περιοχή, όπως δηλαδή και στη μέθοδο των επιτρεπόμενων τάσεων. Έτσι, οι στόχοι του σχεδιασμού είναι: 1) η αποφυγή της κατάρρευσης για τις οριακές καταστάσεις αστοχίας και 2) η παράλληλη ικανοποίηση των απαιτήσεων των οριακών καταστάσεων λειτουργικότητας.

Το πρόβλημα που ανακύπτει για τους οριακούς ελέγχους αστοχίας είναι η υιοθέτηση της κατάλληλης μεθόδου ανάλυσης που θα παρακολουθήσει το φορέα στην οριακή κατάσταση αστοχίας πριν την κατάρρευση. Αν και σήμερα υπάρχουν μέθοδοι που πραγματοποιούν την ανάλυση των φορέων στην πλαστική περιοχή, οι μέθοδοι αυτές βρίσκονται ακόμη σε ερευνητικό επίπεδο και αντιμετωπίζουν κάποια προβλήματα. Οι μέθοδοι που συνήθως ακολουθούνται χρησιμοποιούν προσομοίωση με πεπερασμένα στοιχεία σε μικροσκοπικό επίπεδο (micro modelling), ή και μακροσκοπική προσομοίωση με κατάλληλη ενσωμάτωση της πλαστικής συμπεριφοράς σε προεπιλεγμένες θέσεις (macro modelling). Μεταξύ των προβλημάτων είναι για παράδειγμα οι σημειακές πλαστικές αρθρώσεις. Αυτές θεωρείται ότι προσομοιώνουν ικανοποιητικά τη μετελαστική συμπεριφορά μεταλλικών κατασκευών αλλά όχι κατασκευών από οπλισμένο σκυρόδεμα, όπου η πλαστική συμπεριφορά εκτείνεται σε μία ολόκληρη περιοχή στα άκρα των στοιχείων. Επίσης ο μηχανισμός πλαστικοποίησης υπό διπλή κάμψη με αξονική δύναμη για το σκυρόδεμα δεν είναι μέχρι σήμερα απόλυτα τεκμηριωμένος.

Την αδυναμία αυτή οι περισσότεροι κανονισμοί την καλύπτουν με την εισαγωγή του συντελεστή συμπεριφοράς q με βάση τη σχέση:

$$\Delta_{\pi\lambda} = q\,\Delta_{\varepsilon\lambda}$$

όπου $\Delta_{\pi\lambda}$ είναι οι μετακινήσεις της πραγματικής κατασκευής στην κατάσταση πριν την κατάρρευση και $\Delta_{\varepsilon\lambda}$ είναι οι μετακινήσεις της ελαστικής κατασκευής που αναπτύσσει την ίδια ένταση.

Ενα άλλο βασικό χαρακτηριστικό των κανονισμών συνολικής αντοχής είναι η αντικατάσταση των συντελεστών ασφαλείας της μεθόδου των επιτρεπόμενων τάσεων με τους μερικούς συντελεστές (partial coefficients). Επίσης οι συνδυασμοί φορτίσεων προκύπτουν συστηματικότερα με βάση τη θεωρία αξιοπιστίας, με την οποία προσδιορίζονται και οι μερικοί συντελεστές.

Επιπλέον, οι διατάξεις ενός κανονισμού θα πρέπει να είναι κατά το δυνατόν οι απλούστερες που ανταποκρίνονται στα υπάρχοντα επιστημονικά συμπεράσματα και θεωρίες, για να μειωθεί η πιθανότητα λαθών κατά την εφαρμογή των διατάξεων.

Μεγάλη σημασία έχει για έναν κανονισμό η μαθηματική διατύπωση των διατάξεων (safety format). Στην κατεύθυνση αυτή, οι συστάσεις της διεθνούς επιτροπής ασφάλειας των κατασκευών (International Joint Committee on Structural Safety) [1], [2] είναι ιδιαίτερης σημασίας.

Μαθηματική διατύπωση των οριακών καταστάσεων

Η γενική σχέση που εκφράζει μία οριακή κατάσταση μπορεί να εκφραστεί ως εξής:

$$f\left(x_1, x_2, \ldots, x_n\right) = f(\overline{x}) > 0$$

όπου x_i είναι τυχαίες μεταβλητές που υπεισέρχονται στον προσδιορισμό της οριακής κατάστασης, ενώ f είναι η συνάρτηση της οριακής κατάστασης που για να ικανοποιείται πρέπει να είναι θετική.

Το διάνυσμα των τυχαίων μεταβλητών μπορεί να περιλαμβάνει μόνιμα φορτία G, μεταβλητά φορτία Q, ιδιότητες των υλικών E, γεωμετρικές παραμέτρους D καθώς και αβεβαιότητες του προσομοιώματος Xm. Ενδέχεται επίσης να υπεισέρχονται στον προσδιορισμό της οριακής κατάστασης μία ή και περισσότερες σταθερές C. Έτσι η συνάρτηση οριακής κατάστασης μπορεί να εκφραστεί ως εξής:

$$f\left(\overline{G}, \overline{Q}, \overline{E}, \overline{D}, \overline{X}_m, \overline{C}\right) > 0$$

Στις παραπάνω διατυπώσεις οι συναρτήσεις των οριακών καταστάσεων αποτελούν συναρτήσεις διαφόρων τυχαίων μεταβλητών. Για να αποφευχθεί η στοχαστική ανάλυση για τους ελέγχους των οριακών καταστάσεων οι κανονισμοί υποδεικνύουν ντετερμινιστικές τιμές, κατάλληλα υπολογισμένες, ορίζοντας τις *ονομαστικές τιμές* και τις αντίστοιχες *τιμές σχεδιασμού*.

$$f\left(\overline{G}_d, \overline{Q}_d, \overline{E}_d, \overline{D}_d, \overline{X}_{md}, \overline{C}\right) > 0$$

όπου οι ποσότητες με δείκτη d αποτελούν ντετερμινιστικές τιμές σχεδιασμού της αντίστοιχης τυχαίας μεταβλητής. Οι σχέσεις αυτές χρησιμοποιούνται είτε για τον έλεγχο συγκεκριμένης κατάστασης, είτε για τον προσδιορισμό της ελάχιστης τιμής κάποιας γεωμετρικής παραμέτρου. Στη δεύτερη περίπτωση είτε η συνάρτηση της οριακής κατάστασης επιλύεται ως προς τη ζητούμενη παράμετρο, ή, όπως συμβαίνει τις περισσότερες περιπτώσεις που δεν μπορεί η συνάρτηση να επιλυθεί ως προς τη ζητούμενη παράμετρο, χρησιμοποιείται μία επαναληπτική μέθοδος για τον προσδιορισμό της ελάχιστης τιμής της ζητούμενης παραμέτρου.

$$d_d^i = f^i\left(\overline{G}, \overline{Q}_d, \overline{E}_d, \overline{D}_d, \overline{X}_{md}, \overline{C}\right)$$

Σημαντική είναι η θεώρηση των χαρακτηριστικών τιμών και των τιμών σχεδιασμού των διαφόρων μεγεθών. Μία κατασκευή μπορεί να υπόκειται σε κάποια μεταβλητή δυναμική φόρτιση που περιγράφεται από τη τυχαία μεταβλητή Q. Η χαρακτηριστική τιμή της μεταβλητής αναφέρεται ως Q_k. Η κατασκευή όμως διαστασιολογείται για φορτία

$$Q_d = \gamma_Q Q_k$$

όπου γ_Q είναι ο μερικός συντελεστής της μεταβλητής φόρτισης Q. Άν ο συντελεστής γ_Q αυξηθεί κατά ένα ποσοστό π.χ 20% αυτό σημαίνει αντίστοιχη αύξηση της απαιτούμενης ονομαστικής ικανότητας της κατασκευής για να αναλάβει το φορτίο Q. Σημαίνει επίσης και αντίστοιχη αύξηση της πραγματικής ικανότητας της κατασκευής να αναλάβει το φορτίο Q. Η αύξηση αυτή συνεπάγεται αύξηση των διαστάσεων των μελών και άρα και αύξηση των μονίμων φορτίων ιδίου βάρους και άρα και αύξηση του κόστους κατασκευής.

Από την άλλη πλευρά οι αντοχές σχεδιασμού R_d παρέχονται από σχέσεις της μορφής

$$R_d = R_k / \gamma_m$$

όπου R_k η χαρακτηριστική τιμή της αντοχής και γ_m ο μερικός συντελεστής υλικού, ο οποίος διαιρώντας τη χαρακτηριστική αντοχή έχει τις ίδιες επιπτώσεις στο σχεδιασμό, όπως ο πολλαπλασιαστικός συντελεστής των δράσεων γ_Q.

Η εισαγωγή δύο ομάδων μερικών συντελεστών επιβάλλεται για λόγους εξασφάλισης ισοδύναμης αξιοπιστίας σχεδιασμού μεταξύ διαφόρων τύπων κατασκευών ενός κανονισμού, αλλά και την ορθολογική αντιμετώπιση του προβλήματος του συνδυασμού φορτίσεων, ιδιαίτερα στις περιπτώσεις, όπου για ορισμένα μέλη της κατασκευής η συνολική επιρροή οφείλεται στη διαφορά φορτίων ίδιας τάξεως μεγέθους.

Η γενική μορφή με την οποία μπορούν επίσης να διατυπωθούν οι έλεγχοι είναι:

$$S_d < R_d$$

με την οποία συγκρίνοντας οι δράσεις σχεδιασμού με τις αντίστοιχες αντοχές σχεδιασμού.

Εκτός των οριακών ελέγχων οι κανονισμοί περιλαμβάνουν και κατασκευαστικές διατάξεις που έχουν αποδείξει ότι εξασφαλίζουν τις κατασκευές προσδίδοντας πρόσθετη ασφάλεια. Οι διατάξεις αυτές εξαρτώνται από το υλικό κατασκευής του δομήματος και δεν είναι εύκολο να κωδικοποιηθούν σε επιμέρους ενότητες.

Ο Ευρωκώδικας Νο. 8 ακολουθεί τις αρχές της συνολικής αντοχής, αποτελεί δε μια προσπάθεια θεσμοθέτησης ενός ενιαίου πλαισίου αντισεισμικού σχεδιασμού κτιρίων στα πλαίσια της Ευρώπης και αποβλέπει:

- στην αύξηση του επίπεδου αντισεισμικής ασφάλειας των κατασκευών
- στην ποιοτική βελτίωση της αντισεισμικής δόμησης των κατασκευών.

Ο Ευρωκώδικας Νο. 8 όμως δεν είναι αυτοτελής κανονισμός. Παρέχει κυρίως πρόσθετες διατάξεις που θα πρέπει να πληρούν κατασκευές σε σεισμικές περιοχές και συμπληρώνει κάποιον άλλο Ευρωκώδικα ανάλογα με το υλικό κατασκευής, ενώ σε μερικές περιπτώσεις τροποποιεί ορισμένες διατάξεις άλλων Ευρωκωδίκων.

ΤΕΧΝΗΤΗ ΝΟΗΜΟΣΥΝΗ ΚΑΙ ΕΜΠΕΙΡΑ ΣΥΣΤΗΜΑΤΑ

Τεχνητή νοημοσύνη είναι ο κλάδος της πληροφορικής που μελέτα μεθόδους που επιτρέπουν στους υπολογιστές να διαχειρίζονται καταστάσεις για τις οποίες οι άνθρωποι θεωρούνται καλύτεροι. Τμήμα της τεχνητής νοημοσύνης είναι και η περιοχή των έμπειρων συστημάτων που αναπτύσσει μεθόδους που επιδιώκουν την απόδοση γνώσης που δεν είναι εύκολο να αποδοθεί χρησιμοποιώντας κάποια θεωρία. Τα έμπειρα συστήματα προσφέρονται για την κωδικοποίηση εμπειρικής γνώσης, χρήσιμης για την επίλυση πολλών προβλημάτων, καθώς και για την απόδοση, με τη μορφή πληροφοριακού συστήματος, του τρόπου με τον οποίο εμπειρογνώμονες επιλύουν προβλήματα συγκεκριμένου γνωστικού αντικειμένου.

Η δομή ενός έμπειρου συστήματος αποτελεί εξέλιξη των τρόπων διαχείρισης των πληροφοριών με βάσεις δεδομένων. Κύριο χαρακτηριστικό των βάσεων δεδομένων είναι ότι αναπτύσσονται ανεξάρτητα από το σύστημα επεξεργασίας της πληροφορίας. Το ίδιο ισχύει και για τα έμπειρα συστήματα και έτσι επιπλέον είναι δυνατή και η ανάπτυξη προγραμματιστικών περιβαλλόντων για την ανάπτυξη των έμπειρων συστημάτων (expert system shells).

Η ορολογία που καθιερώθηκε για τα έμπειρα συστήματα ονομάζει τη βάση δεδομένων ως *βάση γνώσης* (knowledge base), για να υποδηλώσει το δυναμικό χαρακτήρα αναπροσαρμογής των δεδομένων. Η μεθοδολογία επεξεργασίας της πληροφορίας αναφέρεται ως *συμπερασματικός μηχανισμός* (inference engine), γιατί χρησιμοποιείται στην εξαγωγή συμπερασμάτων. Έτσι ένα έμπειρο σύστημα αποτελείται κυρίως από τη βάση γνώσης και το συμπερασματικό μηχανισμό.

Η βάση γνώσης μπορεί να αναπτυχθεί χρησιμοποιώντας διάφορες δομές. Μία απλή αναπαράσταση της γνώσης μπορεί να γίνει με κανόνες της μορφής IF...THEN...ELSE. Μπορούν όμως να χρησιμοποιηθούν και συνθετότεροι τρόποι αναπαράστασης της γνώσης με τη μορφή πλαισίων (frame representation). Ο συμπερασματικός μηχανισμός αφορά τον τρόπο, με τον οποίο διερευνάται η δομή ενός δέντρου καταστάσεων. Άν η αναζήτηση ξεκινήσει από τη ρίζα προς τα φύλλα του δέντρου αναφερόμαστε σε μία οπισθοδρομική διαδικασία αναζήτησης συμπερασμάτων (backward chaining), ενώ όταν ξεκινάμε από τα φύλλα προς τη ρίζα αναφερόμαστε σε μία εμπροστοδρομική διαδικασία αναζήτησης συμπερασμάτων (forward chaining).

Στην αναζήτηση αυτή ανακύπτει πολλές φορές το πρόβλημα της επιλογής ενός δρόμου από τους πολλούς που μπορούν να ακολουθηθούν. Απαιτούνται τότε ειδικοί κανόνες για την *επίλυση της σύγκρουσης μεταξύ κανόνων* (conflict resolution). Σημαντική επίσης είναι η δυνατότητα που παρέχουν πολλά έμπειρα συστήματα στο να διαχειρίζονται την αβεβαιότητα στην εξαγωγή των συμπερασμάτων.

Τα τελευταία χρόνια η βιβλιογραφία για τα έμπειρα συστήματα είναι πλούσια σε άρθρα, βιβλία καθώς και σε περιβάλλοντα ανάπτυξης έμπειρων συστημάτων. Παρόλο λοιπόν που η ανάπτυξη ενός έμπειρου συστήματος δεν παρουσιάζει ιδιαίτερες προγραμματιστικές δυσκολίες, η ανάπτυξη ενός έμπειρου συστήματος που να είναι σε θέση να αποδώσει τη γνώση σε μία περιοχή είναι ένα σύνθετο πρόβλημα που εξαρτάται από τη μορφή της γνώσης που το έμπειρο σύστημα καλείται να περιγράψει. Έτσι είναι σημαντικό, προκειμένου να επιλέξει κανείς τα έμπειρα συστήματα ως βάση για την ανάπτυξη ενός προβλήματος, να γνωρίζει άν το πρόβλημα του προσιδιάζει στα ιδιαίτερα χαρακτηριστικά των έμπειρων συστημάτων. Πολλές φορές "έμπειρα συστήματα" θα μπορούσαν να αναπτυχθούν πολύ καλύτερα χρησιμοποιώντας μία αλγοριθμική γλώσσα.

Έτσι, θεμελιώδεις ιδιότητες ενός έμπειρου συστήματος είναι :

- να εφαρμόζεται σε μια συγκεκριμένη περιοχή εμπειρίας.
- να μπορεί να βγάζει συμπεράσματα από αβέβαια δεδομένα.
- να μπορεί να εξηγήσει το δρόμο συλλογισμού του κατά κατανοητό τρόπο .

Σημαντικοί παράγοντες για την επιλογή των έμπειρων συστημάτων στην επίλυση ενός προβλήματος είναι :

- η αβεβαιότητα των δεδομένων
- η μη ύπαρξη καθιερωμένης θεωρίας
- η ύπαρξη πολλών μεταβλητών και αλληλοσυγκρουόμενων παραμέτρων
- οι εμπειρογνώμονες να είναι σπάνιοι και η αμοιβή τους υψηλή

Οι δυνατότητες που παρέχουν τα έμπειρα συστήματα είναι:

- χρήση γλωσσών φιλικών προς τον χρήστη.
- ευχερέστερος χειρισμός συμβόλων.
- διαχωρισμός γνώσης / μηχανισμού ελέγχου -μεγαλύτερη διαφάνεια.
- αναθεώρηση της πορείας συλλογισμού ανάλογα με τα εκάστοτε δεδομένα.
- σταδιακός προγραμματισμός και με αλληλεπίδραση .
- χρήση ευρετικών μεθόδων για περιορισμό του πεδίου έρευνας.
- παράσταση και διαχείριση της γνώσης.
- ευχέρεια στην επέκταση και αναθεώρηση της γνώσης.
- χειρισμός ασαφούς και ατελούς γνώσης.
- δυνατότητα επαγωγικής μάθησης.
- επεξήγηση του δρόμου σκέψης.

Σε αντίθεση τα συμβατικά προγράμματα είναι καλύτερα για προβλήματα που παρουσιάζουν τα παρακάτω χαρακτηριστικά:

- χρήση γλωσσών που βρίσκονται πλησιέστερα στον τρόπο λειτουργίας του υπολογιστή.
- ευχέρεια σε αριθμητικούς υπολογισμούς.
- ανάμιξη δεδομένων (γνώσης) και μηχανισμού ελέγχου - μικρή διαφάνεια.

- προκαθορισμένος δρόμος συλλογισμού.
- ξεχωριστά στάδια σύνταξης, μεταγλώττισης, εκτέλεσης, διόρθωσης.
- χρήση αλγορίθμων.
- παράσταση και χειρισμός δεδομένων.
- η αναθεώρηση της υπάρχουσας γνώσης επιβάλλει ευρείας κλίμακας μεταβολές στο πρόγραμμα.
- δυσχέρεια στο χειρισμό ασαφούς γνώσης.
- δυσχέρεια επαγωγικής μάθησης.

Ως περιορισμοί των σύγχρονων έμπειρων συστημάτων μπορούν να αναφερθούν:

- η απουσία έμπνευσης.
- η έλλειψη ευρύτητας στους συλλογισμούς.
- η έλλειψη προσαρμοστικότητας.
- η δυσχέρεια στη μεταφύτευση σ' αυτά κοινής λογικής.

ΑΝΑΛΥΤΙΚΗ ΠΑΡΟΥΣΙΑΣΗ ΤΗΣ ΔΟΜΗΣ ΤΟΥ ΕΜΠΕΙΡΟΥ ΣΥΣΤΗΜΑΤΟΣ

Ενα έμπειρο σύστημα είναι ενα πληροφοριακό σύστημα, το οποίο αναπτύσσεται συνήθως με βάση κάποιο περιβάλλον ανάπτυξης (expert system shell). Το περιβάλλον αυτό αποτελείται από το βασικό εργαλείο για την ανάπτυξη του συστήματος και από ενα σύνολο βοηθητικών προγραμμάτων διασύνδεσης με το χρήστη. Τα επιμέρους προγράμματα που συνιστούν το έμπειρο σύστημα έχουν γραφτεί σε κάποια η κάποιες γλώσσες προγραμματισμού, οι οποίες βρίσκονται κάτω από το λειτουργικό σύστημα και συνεργάζονται μαζί του. Η δομή ενός τυπικού έμπειρου συστηματος περιλαμβάνει τα εξής:

- προγράμματα διασύνδεσης με το χρήστη.
- πυρήνας του έμπειρου συστηματος.
- εργαλείο λογισμικού για την ανάπτυξη .
- γλώσσα προγραμματισμού.
- λειτουργικό σύστημα
- υπολογιστής

Τον πυρήνα ενός έμπειρου συστήματος αποτελούν δυο αυτόνομα κατ' αρχήν τμήματα, τα οποία είναι:

- η βάση γνώσης (knowledge base)
- ο μηχανισμός εξαγωγής συμπερασμάτων (inference engine)

Τα συστήματα που διατηρούν αυτή τη δομή στον πυρήνα τους και τείνουν να διαχωρίσουν πλήρως τη γνώση που αναφέρεται στο πρόβλημα (domain knowledge) από την υπόλοιπη γνώση (γνώση που σχετίζεται με τον τρόπο επίλυσης του προβλήματος) καλούνται "συστήματα βασισμένα στη γνώση" (knowledge based systems). Τα έμπειρα συστήματα ανήκουν στην κατηγορία των συστημάτων των βασισμένων στη γνώση, και χαρακτηρίζονται από το γεγονός ότι η γνώση που περιέχουν αναφέρεται σε πολύ ειδικούς τομείς.

Ο διαχωρισμός της βάσης γνώσης από το μηχανισμό εξαγωγής συμπερασμάτων παρέχει συγκεκριμένα πλεονεκτήματα, βασικότερα από τα οποία είναι η διαφάνεια του προγραμματισμού και η δυνατότητα πρόσθεσης, μεταβολής ή κατάργησης γνώσης, χωρίς να απαιτούνται ευρείας κλίμακας μεταβολές στο πρόγραμμα.

Το περιβάλλον, πάνω στο οποίο θα λειτουργήσει το έμπειρο σύστημα συμπληρώνεται πολλές φορές από βάσεις δεδομένων και συμβατικά προγράμματα, με τα οποία ενδεχόμενα συνεργάζεται το σύστημα, με διάφορα μέσα επικοινωνίας και αλλά περιφερειακά μηχανήματα.

Στην περίπτωση μεγάλων συστημάτων η υλική υποδομή του περιβάλλοντος λειτουργίας του συστήματος δε ταυτίζεται απαραίτητα με εκείνη του περιβάλλοντος ανάπτυξης του. Από πλευράς λογισμικού, το περιβάλλον ανάπτυξης (development environment) περιλαμβάνει συνήθως, εκτός από το εργαλείο λογισμικού, πάνω στο οποίο χτίζεται ο πυρήνας του συστήματος, και συντάκτες βάσης γνώσης, ελεγκτές της συνέπειας της εισαγόμενης γνώσης, βοηθήματα προσομοίωσης (what-if), αυτόματες ερωτήσεις προς τον χρήστη, παρακολούθηση και επεξήγηση της πορείας συλλογισμού, βοηθήματα διόρθωσης λαθών (debugging aids), προγράμματα εισόδου-εξόδου (input-output facilities) κ.λ.π

Η βάση γνώσης

Η βάση γνώσης περιλαμβάνει όλη την ειδική γνώση που περιγράφει το συγκεκριμένο πρόβλημα. Η γνώση που περιέχεται στη βάση γνώσης διακρίνεται σε στατική και δυναμική. Σα στατική γνώση λογίζονται οι περιγραφές των διάφορων αντικειμένων και γεγονότων και οι συσχετίσεις μεταξύ αυτών. Τα γεγονότα μπορούν να διακριθούν σε γενικά και μόνιμα και σε γεγονότα που αναφέρονται στο συγκεκριμένο πρόβλημα. Τα ειδικότερα αυτά γεγονότα σχηματίζουν, σύμφωνα με μια θεώρηση, τη λεγόμενη βάση δεδομένων (data base), ενώ το σύνολο των γεγονότων που περιλαμβάνονται στη βάση γνώσης αποτελούν τη βάση γεγονότων (facts base).

Η δυναμική γνώση, η οποία αποτελείται συνήθως από διαδικασίες και κανόνες, παρέχει τα μέσα για την αποτελεσματική χρησιμοποίηση της στατικής γνώσης για τη δημιουργία νέων γεγονότων. Τα νέα γεγονότα προστίθενται στη βάση γεγονότων, που ουσιαστικά αποτελεί την μνήμη εργασίας του συστήματος (working memory). Στη δυναμική γνώση μπορεί να περιλαμβάνονται και πολύ εξειδικευμένες διαδικασίες που σχετίζονται με το ειδικό πρόβλημα που χειρίζεται το έμπειρο σύστημα, αυτό όμως δεν είναι απόλυτο, δεδομένου ότι τέτοιες διαδικασίες προτιμάται πολλές φορές να ενσωματώνονται στο μηχανισμό εξαγωγής συμπερασμάτων. Από τους γενικούς τρόπους παράστασης της γνώσης, που αναφέρθηκαν προηγούμενα στα έμπειρα συστήματα, χρησιμοποιούνται συνηθέστερα η τυπική λογική, τα σημασιολογικά δίκτυα και τα πλαίσια. Πολλά έμπειρα συστήματα, αλλά και εργαλεία ανάπτυξης, έχουν σήμερα υιοθετήσει τον αντικειμενοστρεφή προγραμματισμό (object oriented programming).

Μια ειδική κατηγορία έμπειρων συστημάτων είναι εκείνη των "συστημάτων που βασίζονται σε κανόνες" (rule based expert systems), των οποίων ο κύριος όγκος της γνώσης παριστάνεται με κανόνες. Πάνω σε αυτή την λογική έχει φτιαχτεί και το πρόγραμμα για τον Ευρωκώδικα No. 8. Η εσωτερική δομή (το προκείμενο και η ενέργεια) των κανόνων περιγράφεται όμως πολλές φορές και με τη χρήση άλλων γλωσσών, όπως η LISP και η PROLOG, οι οποίες, αντίθετα με τα περιβάλλοντα ανάπτυξης, παρουσιάζουν σαφώς μεγαλύτερη ευελιξία και προσαρμοστικότητα, εφόσον δεν στηρίζονται σε κάποιο στερεότυπο σύστημα.

Μ' αυτή τη συλλογιστική οι κανόνες δεν παρουσιάστηκαν σαν ένας ξεχωριστός τρόπος παράστασης σε σχέση με τη λογική, αν και ο φορμαλισμός είναι ασφαλώς συγκρίσιμος με τις δομημένες μεθόδους παράστασης γνώσης, που έχουν προηγούμενα παρατεθεί. Άλλωστε ο διαχωρισμός μεταξύ συστημάτων βασισμένων στη λογική και συστημάτων βασισμένων σε κανόνες, είναι πολύ σχετικός, δεδομένου ότι τα περισσότερα από τα υπάρχοντα συστήματα που βασίζονται στη λογική έχουν αναπτυχθεί με τέτοιο τρόπο, ώστε ο χρήστης να αντιλαμβάνεται μάλλον τους κανόνες παρά τη λογική, η οποία κρύβεται κάτω απ'αυτούς. Ετσι, κανείς δε θα απέκλειε τη ταξινόμηση αυτών των συστημάτων ως βασισμένων σε κανόνες.

Ο προγραμματισμός που βασίζεται στα αντικείμενα εισάγει μια δομημένη θεώρηση και περιγραφή των προβλημάτων. Βασίζεται σε δομημένες οντότητες, οι οποίες καλούνται αντικείμενα (objects). Τα αντικείμενα μπορούν να συσχετίζονται μεταξύ τους με διάφορους τρόπους, να ταξινομούνται σε γενικότερες κατηγορίες, να εμπεριέχουν δεδομένα και διαδικασίες, να επικοινωνούν μεταξύ τους με μηνύματα και να κληρονομούν αυτόματα ιδιότητες από άλλα αντικείμενα. Τα αντικείμενα αποτελούν δομές δεδομένων που ενσωματώνουν αφενός τα δεδομένα που χαρακτηρίζουν το αντικείμενο, αλλά και τις μεθόδους με τις οποίες αυτά αντιδρούν σε δεδομένα μηνύματα. Η εκτέλεση ενός προγράμματος βασισμένου σε αντικειμενοστρεφή προγραμματισμό συνίσταται στην ανταλλαγή μηνυμάτων που ενεργοποιούν αντίστοιχες μεθόδους. Η φιλοσοφία αυτού του προγραμματισμού βασίζεται ουσιαστικά στον τρόπο, με τον οποίο ο άνθρωπος συλλαμβάνει και χειρίζεται τις έννοιες και τα αντικείμενα. Ετσι, αποτελεί ένα ιδιαίτερα φυσικό τρόπο θεώρησης και περιγραφής των προβλημάτων, ο οποίος βρίσκεται αρκετά κοντά στην ανθρώπινη διαίσθηση και εμπειρία. Γι αυτό ακόμη και κλασσικές γλώσσες προγραμματισμού έχουν επεκταθεί, ώστε να υποστηρίζουν παραστάσεις βασισμένες σε αντικείμενα, όπως για παράδειγμα η C++, η Objective C, η Prolog++ και άλλες, ενώ το προγραμματιστικό περιβάλλον των MS-Windows 3.x βασίζεται επίσης σε αντικείμενα.

Συχνά για την περιγραφή ενός προβλήματος παρατηρείται στην πράξη η χρησιμοποίηση συγχρόνως περισσοτέρων του ενός τρόπων παράστασης της γνώσης. Σ' αυτή την περίπτωση η αρχιτεκτονική του συστήματος καλείται υβριδική (hybrid architecture). Σε ένα υβριδικο σύστημα η γνώση παριστάνεται συνήθως με την μορφή κανόνων, οι οποίοι αποτελούν το δυναμικό μέρος της γνώσης, καί με τη χρήση δομημένων παραστάσεων (δομημένες περιγραφές αντικείμενων).

Ανεξάρτητα όμως από το συγκεκριμένο τρόπο παράστασης της γνώσης που χρησιμοποιείται, θα πρέπει να εξεταστεί σοβαρά και το επίπεδο λεπτομέρειας της παράστασης που θα υιοθετεί. Γεννάται δηλαδή το ερώτημα εάν η περιγραφή της γνώσης είναι σκόπιμο να γίνει μέσω μιας ομάδας πρωτογενών στοιχείων χαμηλού επίπεδου, ή με χρήση τρόπων έκφρασης υψηλότερου επίπεδου. Υπενθυμίζεται ότι η χρήση χαμηλού επίπεδου παράστασης οδηγεί σε πολύπλοκη υπολογιστική εργασία για την παράσταση πιο σύνθετης γνώσης, καθώς επίσης και σε σαφώς ογκωδέστερη βάση γνώσης. Επιπλέον, η επιλογή των πρωτογενών εκείνων στοιχείων που θα διευκολύνουν μια ικανοποιητική παράσταση της γνώσης σε ένα συγκεκριμένο πρόβλημα, είναι σε πολλές περιπτώσεις εξαιρετικά δυσχερής. Βέβαια η χρήση ενός περιορισμένου αριθμού πρωτογενών στοιχείων συνεπάγεται ένα μονοσήμαντο και σαφέστερα καθορισμένο τρόπο παράστασης. Οι μηχανισμοί που απαιτούνται για την εξαγωγή συμπερασμάτων είναι περισσότερο σαφείς, αυστηροί, και ολιγάριθμοι, δεδομένου ότι αναφέρονται μόνο στα ολιγάριθμα πρωτογενή στοιχεία.

Ο μηχανισμός εξαγωγής συμπερασμάτων

Ο μηχανισμός εξαγωγής συμπερασμάτων είναι το τμήμα εκείνο του συστήματος, το οποίο περιέχει τη γνώση που αναφέρεται στο χειρισμό και έλεγχο της βάσης και ασχολείται με την εύρεση της λύσης του προβλήματος. Αποτελεί δηλαδή, το μηχανισμό σκέψης του συστήματος. Η δομή του μηχανισμού εξαγωγής συμπερασμάτων εξαρτάται άμεσα από τον τρόπο, με τον οποίο αναπαρίσταται και οργανώνεται η γνώση μέσα στη βάση της γνώσης και από τις ιδιομορφίες του συγκεκριμένου προβλήματος. Η ποιότητα του συνδέεται άρρηκτα με την ικανότητα του για αποτελεσματικό χειρισμό της βάσης γνώσης και γρήγορο εντοπισμό της λύσης. Ο μηχανισμός εξαγωγής συμπερασμάτων μπορεί να διαχωριστεί σε δυο μέρη, το πρώτο ονομάζεται διερμηνέας (interpreter) και το δεύτερο χρονοσχεδιαστής η επιλογέας (scheduler). Ο διερμηνέας είναι ο μηχανισμός που χειρίζεται την υπάρχουσα γνώση με σκοπό την εξαγωγή νέας γνώσης. Εφαρμόζει συγκεκριμένες τεχνικές εξαγωγής συμπερασμάτων (inference techniques), όπως είναι οι στατιστικές μέθοδοι και οι μέθοδοι αναγνώρισης προτύπων (pattern matching). Με τη χρησιμοποίηση αυτών των μεθόδων εκτιμώνται οι προτάσεις, τα γεγονότα, οι σχέσεις και οι κανόνες που απαρτίζουν τη βάση γνώσης. Ετσι, για παράδειγμα σε ένα σύστημα βασισμένο σε κανόνες, ο διερμηνέας εντοπίζει τους κανόνες, των οποίων είναι δυνατή η εφαρμογή σε μια δεδομένη στιγμή. Ο χρονοσχεδιαστής ή επιλογέας αποφασίζει πότε και με ποια σειρά θα χρησιμοποιηθούν τα διάφορα στοιχεία της βάσης γνώσης. Περιέχει γνώση εκτίμησης (assessment knowledge), η οποία εκτιμά τους εναλλακτικούς δρόμους έρευνας και γνώση ελέγχου (control knowledge), η οποία συντονίζει την ενεργοποίηση και τον προγραμματισμό των λειτουργιών. Ο επιλογέας εκφράζει δηλαδή τη στρατηγική ελέγχου του συστήματος.

Ένα μείζον πρόβλημα στην Τεχνητή Νοημοσύνη αποτελεί η επιλογή του στοιχείου της γνώσης, το οποίο είναι το πλέον κατάλληλο να εφαρμοστεί στο κάθε βήμα της πορείας επίλυσης, όταν υπάρχουν συγχρόνως πολλά υποψήφια για εφαρμογή στοιχεία. Το πρόβλημα αυτό είναι γνωστό σαν επίλυση της σύγκρουσης (conflic resolution). Μια γενική αρχή που εφαρμόζεται από τις διάφορες στρατηγικές που ασχολούνται με την αντιμετώπιση αυτού του προβλήματος, συνίσταται στην απόδοση της υψηλότερης προτεραιότητας στις προτάσεις που είναι περισσότερο περιοριστικές. Η απαίτηση αυτή είναι απόλυτα κατανοητή, δεδομένου ότι ο έλεγχος που ξεκινά από τελείως γενικές προτάσεις για να προχωρήσει σε ειδικότερες, είναι εύκολο να οδηγήσει σε σπατάλη υπολογιστικού χρόνου. Στρατηγικές που εφαρμόζονται σε συστήματα βασισμένα σε κανόνες είναι:

- Χορήγηση προτεραιότητας σε κανόνες με ειδικότερο προκείμενο, δεδομένου ότι αυτοί οι κανόνες δίνουν συνήθως ακριβέστερα συμπεράσματα.

- Χορήγηση προτεραιότητας σε κανόνες που δίνουν τον μεγαλύτερο αριθμό συμπερασμάτων.

- Χρήση μετα-κανόνων (meta-rules), που χειρίζονται άλλους κανόνες. Οι μετα-κανόνες υποδεικνύουν τον κανόνα που θεωρείται πιο πρόσφορο να εφαρμοστεί σε μια δεδομένη στιγμή.

Η στρατηγική ελέγχου είναι πολλές φορές αναγκαίο να περιέχει γνώση που αναφέρεται στο συγκεκριμένο πρόβλημα, το οποίο χειρίζεται, και μάλιστα σε έκταση όση επιβάλλουν οι ιδιομορφίες του προβλήματος. Παρατηρείται, δηλαδή, η μεταφορά ειδικής γνώσης από τη βάση γνώσης στο μηχανισμό ελέγχου. Αυτό έρχεται σε κάποια αντίθεση με την εικόνα που έχει δοθεί περί ύπαρξης πλήρους και απόλυτου διαχωρισμού μεταξύ βάσης και μηχανισμού εξαγωγής συμπερασμάτων. Ετσι στις περισσότερες περιπτώσεις ο μηχανισμός εξαγωγής συμπερασμάτων περιέχει και γνώση που σχετίζεται με το πρόβλημα, οπότε και χαρακτηρίζεται σαν εξαρτημένος από το συγκεκριμένο πεδίο γνώσης (domain dependent).

Περίπτωση μίξης ειδικής γνώσης και γνώσης ελέγχου έχει ήδη αναφερθεί στις περιπτώσεις της διαδικαστικής προσάρτησης και στα πλαίσια της διαδικαστικής αναπαράστασης. Εκεί, η μίξη αυτή γίνεται στη βάση της γνώσης, η οποία πλέον περιέχει, εκτός από την ειδική γνώση του προβλήματος, και κάποιο κομμάτι γνώσης που σχετίζεται με τον τρόπο επίλυσης του προβλήματος.

Υπάρχουν δυο χαρακτηριστικές μέθοδοι που εφαρμόζονται κατά την έρευνα στο πεδίο καταστάσεων για τον εντοπισμό της λύσης ενός προβλήματος, και συγκεκριμένα οι μέθοδοι εμπροστοδρομικής αναζήτησης (forward reasoning ή forward chaining) και της ανάστροφης συλλογιστικής της οπισθοδρομικής αναζήτησης (backward reasoning η backward chaining). Η μέθοδος της εμπροστοδρομικής αναζήτησης ξεκινά την έρευνα για την ανεύρεση νέας πληροφορίας από την ήδη υπάρχουσα γνώση. Για το λόγο αυτό η μέθοδος αυτή χαρακτηρίζεται σαν καθοδηγούμενη από τα δεδομένα (data driven) ή σαν πορεία που ξεκινά από τη βάση και προχωρά προς το αποδεικτέο (bottom-up). Αυτή η μέθοδος έρευνας είναι φανερό ότι μπορεί εύκολα να αποπροσανατολιστεί από το ζητούμενο εφόσον το σύστημα συλλογίζεται χωρίς να έχει σε κάθε βήμα κάποια ένδειξη, η οποία να το καθοδηγεί προς το αποδεικτέο. Αυτό συμβαίνει επειδή το σύστημα δε λαμβάνει καθόλου υπόψη του το ζητούμενο κατά την πορεία συλλογισμού. Ετσι, μόνο κατά τύχη μπορεί να καταλήξει στο ζητούμενο, εκτός βέβαια αν βοηθηθεί από κατάλληλες ευρετικές μεθόδους. Γι' αυτό το λόγο πολλές φορές χρησιμοποιείται η οπισθοδρομική αναζήτηση, η οποία ξεκινά από το αποδεικτέο και προσπαθεί να καταλήξει σε ήδη γνωστά γεγονότα. Για τούτο ονομάζεται επίσης μέθοδος προσανατολισμένη στο αποδεικτέο (goal oriented) ή εκ των άνω προς τα κάτω (top-down). Τίθεται πλέον το ερώτημα ποιά από τις δύο παραπάνω μεθόδους είναι η καταλληλότερη για τη λύση ενός προβλήματος. Σε προβλήματα, στα οποία ο αριθμός των δυνατών επιλογών σε κάθε βήμα είναι ο ίδιος και για τις δυο μεθόδους και το πλήθος των δυνατών αρχικών καταστάσεων είναι το αυτό με το πλήθος των αποδεικτέων, δεν υπάρχουν προφανή επιχειρήματα που να ενισχύουν την επιλογή της μιας η της άλλης μεθόδου. Τέτοια προβλήματα όμως αποτελούν μάλλον την εξαίρεση και όχι τον κανόνα. Ετσι η "τοπολογία" του πεδίου έρευνας είναι στις περισσότερες περιπτώσεις καθοριστική για την επιλογή μεταξύ των δυο μεθόδων.

Η εμπροστοδρομική συλλογιστική είναι περισσότερο κατάλληλη για προβλήματα που χαρακτηρίζονται από μικρότερο αριθμό αρχικών καταστάσεων απ' ότι αποδεικτέων, όπως και για προβλήματα, στα οποία η ορθή συλλογιστική γεννά μικρότερο αριθμό δυνατών επιλογών σε κάθε βήμα της πορείας επίλυσης, απ' όσες θα δημιουργούσε η εφαρμογή της οπισθοδρομικής συλλογιστικής. Σε αντίθετη περίπτωση, όταν τα αποδεικτέα είναι λιγότερα από τις δυνατές καταστάσεις και όταν η εφαρμογή της οπισθοδρομικής συλλογιστικής οδηγεί σε μικρότερο αριθμό δυνατών επιλογών σε κάθε βήμα, τότε η συλλογιστική αυτή είναι περισσότερο πρόσφορη, βοηθά δε συνήθως στη "μηχανική" επεξηγηματική ικανότητα του συστήματος. Ενα κλασσικό παράδειγμα εφαρμογής της οπισθοδρομικής συλλογιστικής αποτελεί η απόδειξη θεωρημάτων. Άλλα τυπικά προβλήματα, στα οποία χρησιμοποιείται η συλλογιστική αυτή είναι τα προβλήματα διαγνωστικής και παρακολούθησης παραμέτρων λειτουργίας.

Σε μερικά συστήματα εφαρμόζεται συγχρόνως η ορθή και η ανάστροφη συλλογιστική. Στην περίπτωση αυτή η εκκίνηση γίνεται και από τα δεδομένα προς τα εμπρός και από τα αποδεικτέα προς τα πίσω, έως ότου οι δυο αυτές πορείες συναντηθούν. Η μέθοδος αυτή ονομάζεται αμφίδρομη συλλογιστική (bi-directional reasoning). Το βασικό πρόβλημα που αντιμετωπίζεται κατά την εφαρμογή αυτής της μεθόδου είναι η πιθανότητα να μη συναντηθούν οι δυο πορείες. Για την αντιμετώπιση του προβλήματος αυτού εφαρμόζονται ειδικές ευρετικές μέθοδοι.

Η μέθοδος οπισθοδρόμησης χρησιμοποιείται στην περίπτωση που το σύστημα φθάνει σε ενα σημείο, από το οποίο είναι επιθυμητό να επιστρέψει σε προηγούμενες καταστάσεις για να συνεχίσει την έρευνα του προς άλλη κατεύθυνση. Πρόκειται ακριβώς για κάτι ανάλογο με τη μέθοδο που ακολουθεί ένας πεζοπόρος που προσπαθεί να εντοπίσει μια συγκεκριμένη τοποθεσία μέσα σε ένα πυκνό και άγνωστο δάσος. Ξεκίνα από ένα γνωστό του σημείο προς την πλέον πιθανή κατεύθυνση, σημειώνοντας καλά στην μνήμη του το δρόμο, έως ότου διάφορες παρατηρήσεις ή μαρτυρίες τον πείσουν ότι δεν ακολουθεί το σωστό δρόμο. Τότε επιστρέφει στο γνωστό σημείο εκκίνησης και επιλέγει άλλη κατεύθυνση έρευνας, που σύμφωνα με ορισμένα κριτήρια είναι η περισσότερο πιθανή να οδηγήσει σε επιτυχές αποτέλεσμα. Το γεγονός ότι η οπισθοδρόμηση αποτελεί μια συνηθέστατα απαντώμενη τεχνική της ανθρώπινης σκέψης, την έχει καταστήσει ιδιαίτερα χρήσιμη και στα έμπειρα συστήματα.

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ΚΑΝΟΝΙΣΜΟΙ ΩΣ ΕΜΠΕΙΡΑ ΣΥΣΤΗΜΑΤΑ

Οι κανονισμοί περιέχουν γενικές οδηγίες για την προσομοίωση τις μεθόδους ανάλυσης και τους ελέγχους συνολικής αντοχής που απαιτούνται για να εξασφαλιστεί ο σχεδιασμός ενός δομήματος με την αξιοπιστία που επιβάλει το σημερινό επίπεδο γνώσεων. Περιλαμβάνονται επίσης και κατασκευαστικές πρακτικές που έχουν επικρατήσει στο σχεδιασμό των δομημάτων ανάλογα με το υλικό κατασκευής.

Η προσπάθεια διαχείρισης των κανονισμών με τη μορφή πληροφοριακών συστημάτων έχει ξεκινήσει από τη δεκαετία του 1960 στις ΗΠΑ από τον S. Fenves και άλλους [3]. Στη κατεύθυνση αυτή βασικές θεωρούνται οι εργασίες των S. Fenves και Liadis [4] και Fenves και Wright [5]. Ο S. Fenves μετείχε επίσης στην ομάδα σύνταξης του κανονισμού μεταλλικών έργων της AISC (LRFD) ως υπεύθυνος για την λογισμιτικοποίηση της ροής των ελέγχων του κανονισμού. Οι Rosenman and Gero [6] ανέπτυξαν τον Κτιριοδομικό κανονισμό της Αυστραλίας με τη μορφή έμπειρου συστήματος βασισμένου σε κανόνες. Με την μορφή έμπειρου συστήματος έχει αναπτυχθεί και ο κανονισμός πυροπροστασίας της Νέας Ζηλανδίας (Buis [7]).

Σημαντικές στην περιοχή αυτή θεωρούνται και οι εργασίες των Garrett και Fenves [8] και [9] και των Harris και Wright [10]. Οι Kumar, Georges και Topping χρησιμοποιούν προγραμματισμό με αντικείμενα για την παράσταση των κανονισμών [11].

Η γενική δομή των διατάξεων αποτελεί την πρώτη ενότητα που μπορεί να επεξεργαστεί ένα έμπειρο σύστημα. Η δομή αυτή είναι συνήθως δενδροειδής. Η σειρά των γενικών ελέγχων που θα πρέπει να ακολουθηθούν προκύπτει από την ενότητα αυτή.

Μια δεύτερη ενότητα αφορά τους ελέγχους των οριακών καταστάσεων που επιβάλει ο κανονισμός ανάλογα με το υλικό κατασκευής του έργου, καθώς και τις κατασκευαστικές διατάξεις που θα πρέπει να τηρηθούν.

Σε μία τρίτη ενότητα θα μπορούσαν να διαταχθούν οι απαιτήσεις και η σειρά των υπολογισμών για κάθε οριακό έλεγχο. Η ενότητα αυτή είναι η πλέον αλγοριθμική και έχει αποτελέσει το αντικείμενο έρευνας από τη δεκαετία του 70 με κύρια τη συνδρομή του S. Fenves και των λογικών πινάκων ενεργοποίησης των ελέγχων.

Ο Ευρωκώδικας Νο. 8 αποτελεί συμπληρωματικό κανονισμό στους αντίστοιχους κανονισμούς που καλύπτουν τη διαστασιολόγηση κατασκευών με βάση συγκεκριμένο υλικό. Έπομένως δεν είναι εφικτή η ανεξάρτητη ανάλυση της ενότητας αυτής χωρίς τους άλλους Ευρωκώδικες. Έτσι η παρούσα εργασία δεν αντιμετωπίζει τον κανονισμό αλγοριθμικά αλλά ως περιγραφή των αντίστοιχων απαιτήσεων.

ΚΑΝΟΝΙΣΜΟΙ ΩΣ ΣΥΣΤΗΜΑΤΑ HYPERTEXT

Τα συστήματα Hypertext αποτελούν δενδροειδείς δομές, οι κόμβοι των οποίων μπορούν να ενεργοποιούν αρχεία με κείμενα, εικόνες, video καθώς και άλλα προγράμματα. Τα συστήματα Hypertext αναπτύσσονται και αυτά σε κατάλληλα περιβάλλοντα. Σήμερα τα Windows 3.x, με όλα τα πλεονεκτήματα του αντικειμενοστρεφούς προγραμματισμού, παρέχουν τα εργαλεία για την ανάπτυξη συστημάτων Hypertext με τις ιδιότητες των προγραμμάτων "Help" των εφαρμογών για Windows 3.x.

Η δενδροειδής δομή του κανονισμού αναπτύσσεται με τη μορφή συστήματος Hypertext που δυναμικά συνδέει κείμενα, φράσεις ή εικόνες κεφαλαίων του κανονισμού με επεξηγηματικά κείμενα ή εικόνες που και αυτά μπορούν να είναι Hypertext. Με τον τρόπο αυτό αναδεικνύεται η διαπλοκή των διατάξεων του κανονισμού και παρέχονται οι απαραίτητες εξηγήσεις. Στην παρούσα εργασία έχουν αναπτυχθεί όλες οι διατάξεις του Ευρωκώδικα όπως παρουσιάζονται στο σχέδιο ?????. Η μορφή αυτή του Ευρωκώδικα δεν είναι η οριστική αλλά εύκολα θα μπορούσε κανείς να ενσωματώσει τις αλλαγές.

Παρόλο που τα συστήματα Hypertext αναπτύσσονται σε ειδικά περιβάλλοντα, η δομή των συστημάτων αυτών ακολουθεί την αναζήτηση της πληροφορίας όπως αυτή γίνεται στις βάσεις δεδομένων και παραπέρα στα έμπειρα συστήματα. Έτσι η συμπεριφορά των συστημάτων αυτών εμφανίζει τα ίδια χαρακτηριστικά με τα έμπειρα συστήματα, αν και από δομικής πλευράς δεν ταυτίζεται με τη δομή των τυπικών έμπειρων συστημάτων με την στενή έννοια.

Προτεινόμενη δομή ανάπτυξης του συστήματος

Για τη βελτίωση της χρήσης ενός κανονισμού στη παρούσα εργασία θεωρείται ότι οι βασικές αρχές ενός κανονισμού συνολικής αντοχής καθώς και η διαπλοκή των βασικών ενοτήτων του κανονισμού είναι γνωστές και αναμενόμενες. Η κυριώτερη δυσκολία εντοπίζεται στο να επιλέξει κανείς με βεβαιότητα και πληρότητα τις διατάξεις που αφορούν τη μελέτη επιμέρους θεμάτων. Έτσι η διαχωριστική γραμμή τίθεται μεταξύ της γενικής και ειδικής γνώσης του κανονισμού. Με βάση τη θεώρηση αυτή κρίθηκε ότι αντίστοιχη θα πρέπει να είναι και η δομή του πληροφοριακού συστήματος για την παρουσίαση του Ευρωκώδικα No. 8. Επιλέχθηκε λοιπόν να παρουσιάζεται στο χρήστη η διασύνδεση των ενοτήτων του κανονισμού με τη μορφή δέντρου, που άλλωστε είναι απλή και με τη συνεχή χρήση προσδίδει την εποπτεία που χρειάζεται και βοηθά να εντυπώνεται στο χρήστη η δομή του κώδικα. Σε αντίθετη περίπτωση, το δέντρο θα παρέμενε στη βάση γνώσης του συστήματος το οποίο με ερωτήσεις θα παρουσίαζε τη ζητούμενη κάθε φορά διαδρομή, δημιουργώντας μία λαβυρινθώδη εντύπωση για τη διαπλοκή του κώδικα.

Το πλέον περίπλοκο θέμα της επιλογής με βεβαιότητα και πληρότητα των διατάξεων που αφορούν τη μελέτη των ειδικότερων θεμάτων μπορεί να αντιμετωπιστεί σε δύο επίπεδα. Πρώτα στο επίπεδο της αναζήτησης των ενεργών διατάξεων που αφορούν το συγκεκριμένο θέμα και την αλληλουχία τους και στη συνέχεια στο αλγοριθμικό επίπεδο όπου ακολουθούνται τα βήματα των απαιτούμενων υπολογισμών που απαιτεί ο κώδικας.

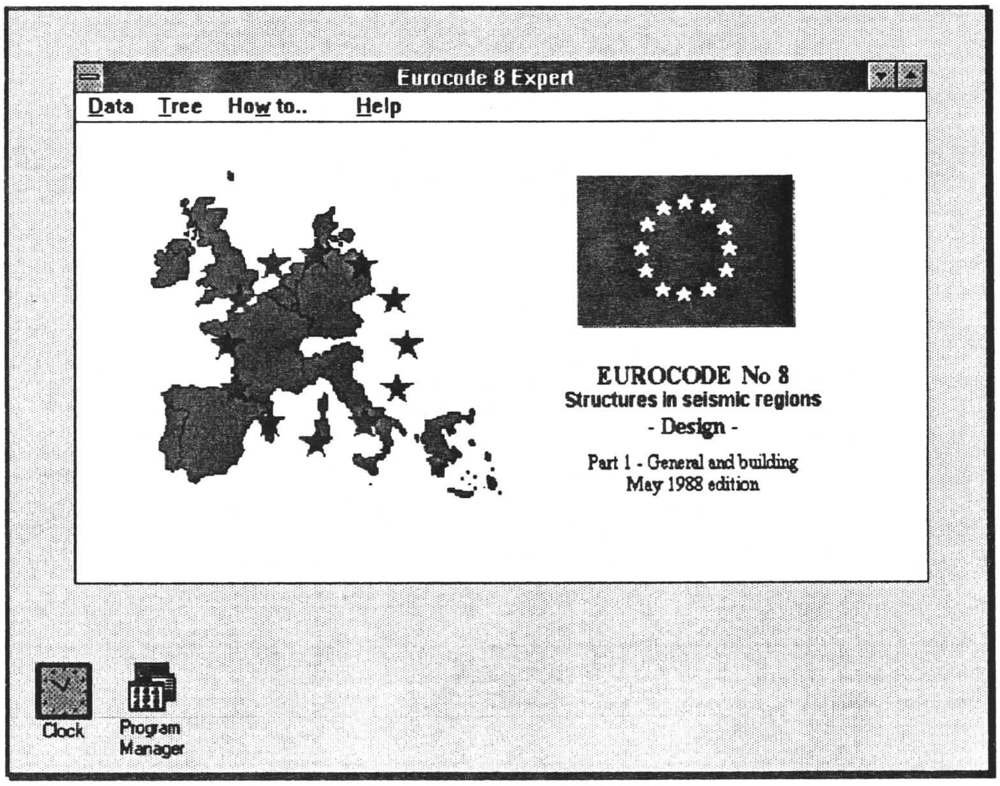
Βασικό θέμα για την ανάπτυξη του συστήματος αποτέλεσε και η επιλογή μεταξύ της μορφής της πληροφορίας και του περιεχόμενου, δηλ. το αν οι διατάξεις του κώδικα θα ακολουθήσουν την καθιερωμένη δομή ενός έμπειρου συστήματος ή θα χρησιμοποιείται η προσφορότερη και αμεσότερη διαθέσιμη φόρμα για την απόδοση του προβλήματος. Επιλέχθηκε η ανάδειξη του περιεχομένου ως σημαντικότερου.

Το όλο σύστημα αν και ξεκίνησε με έμφαση την ανάπτυξή του με τη μορφή κανόνων ως τυπικό έμπειρο σύστημα, μετατοπίστηκε στη εκτεταμένη χρήση hypertext που κρίθηκε ότι αποδίδει πολύ καλύτερα τη γενική φάση και την πρώτη επιμέρους φάση. Το αλγοριθμικό μέρος του κανονισμού σε συνδυασμό με τους αντίστοιχους κώδικες μπορεί ολοκληρωμένα να αναπτυχθεί χρησιμοποιώντας τη γλώσσα Prolog.

ΠΑΡΟΥΣΙΑΣΗ ΤΟΥ ΣΥΣΤΗΜΑΤΟΣ

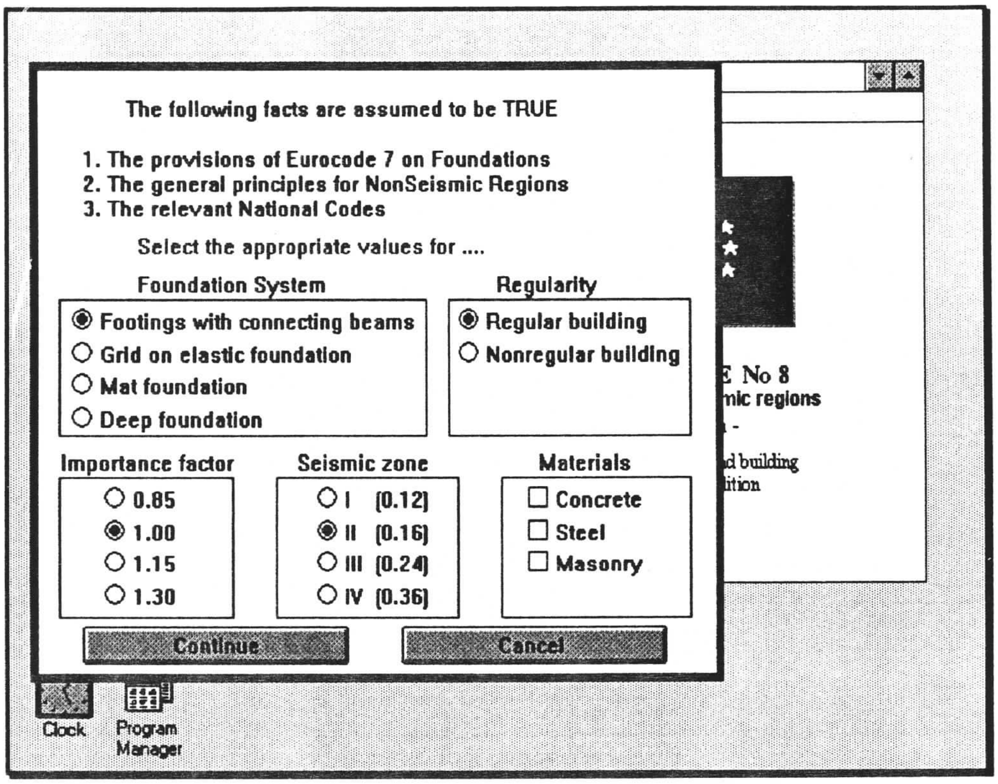
Αρχικές Εντολές

Το σύστημα έχει αναπτυχθεί, όπως αναφέρθηκε, σε περιβάλλον MS-Windows 3.x και ενεργοποιεί τα διάφορα επιμέρους προγράμματα με την κατάλληλη επιλογή από το κύριο menu που παρουσιάζεται στην Εικ. 1. Στο κύριο menu εμφανίζονται οι γενικές πληροφορίες Data, η ενότητα Tree και η ενότητα How to.., σχετικά με το πώς αναλύεται κάποια συγκεκριμένη κατασκευή, ή πώς σχεδιάζεται κάποιο μέλος κατασκευής από συγκεκριμένο υλικό. Υπάρχει επίσης η δυνατότητα ενεργοποίησης των επεξηγήσεων Help. Ολες οι επιλογές αυτές μπορούν να ενεργοποιηθούν με το ποντίκι του υπολογιστή, ή με το πάτημα του πλήκτρου που υπογραμμίζεται.



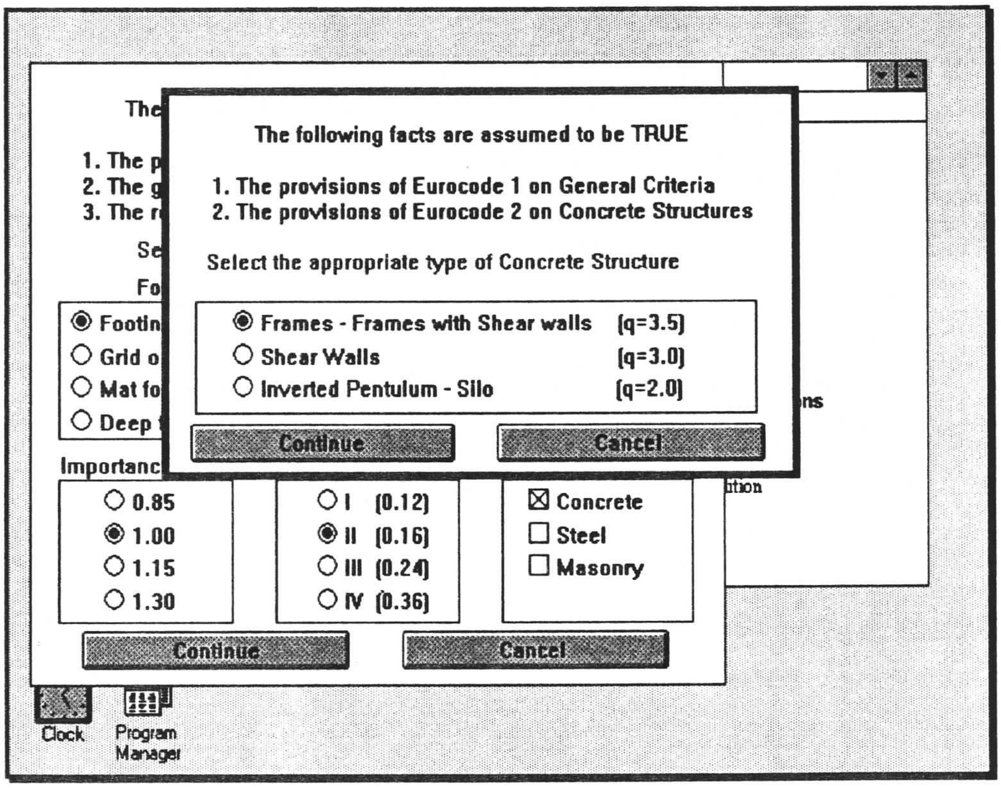
Εικ. 1. Κύριο menu του συστήματος

Στις γενικές πληροφορίες ο χρήστης ενημερώνεται για το τι θεωρείται από το σύστημα ως αληθές για τις παρακάτω ενέργειες ανάλογα με το υλικό κατασκευής. Έτσι στη Εικ. 2 βλέπουμε τις γενικές πληροφορίες που αφορούν το σύστημα θεμελίωσης, την κανονικότητα του κτιρίου, την κοινωνική σπουδαιότητα του δομήματος, τη σεισμική περιοχή καθώς και το υλικό κατασκευής. Οι τιμές που αναγράφονται στη σπουδαιότητα και τη σεισμική ζώνη είναι αυτές του ΝΕΑΚ και αφορούν τον Ελλαδικό χώρο.

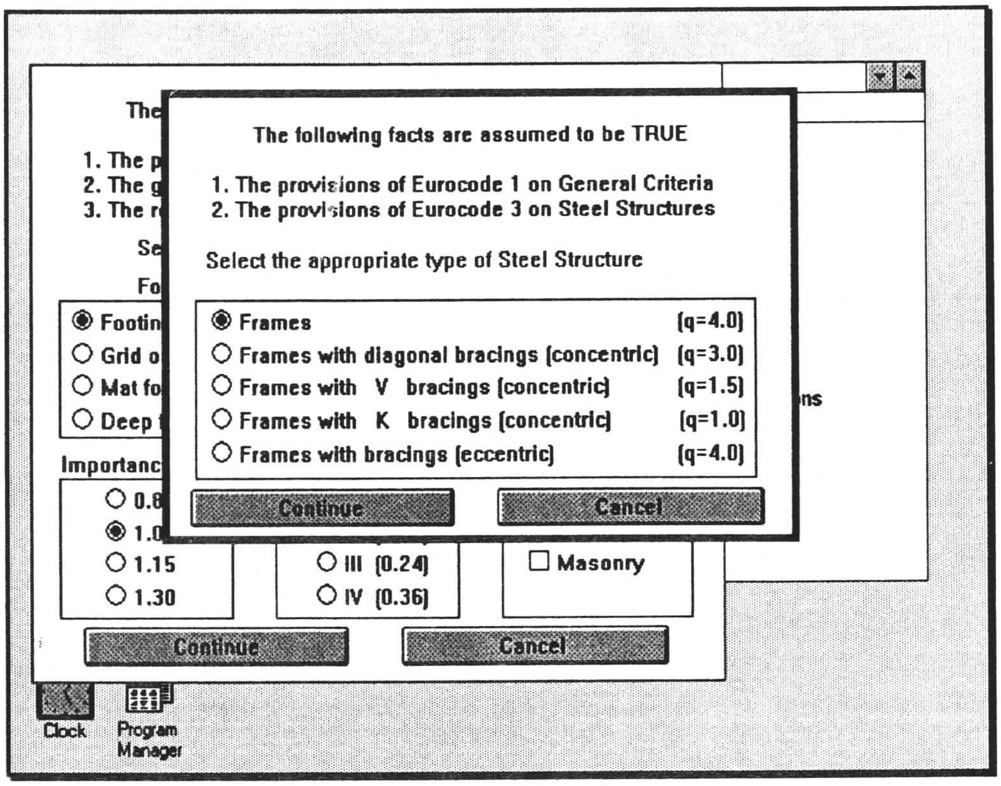


Εικ. 2 Γενικές πληροφορίες για τη κατασκευή

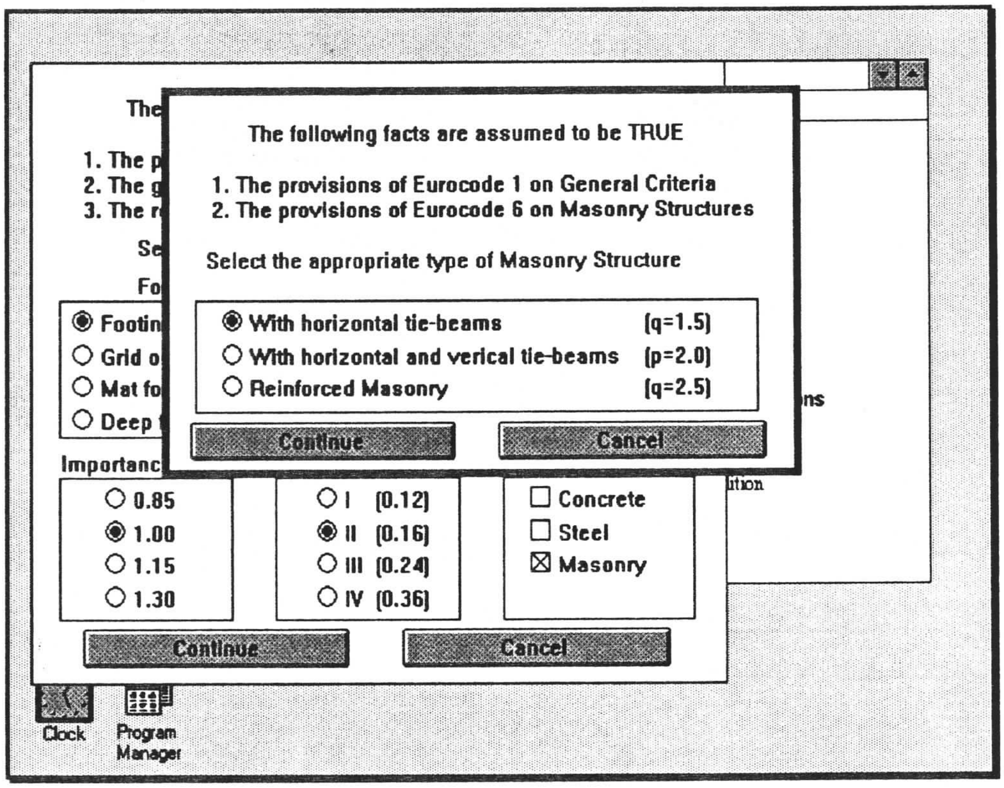
Στις Εικ. 3 και 4 παρουσιάζονται οι γενικές πληροφορίες που αφορούν κατασκευές από σκυρόδεμα και χάλυβα που καλούνται με την ενεργοποίηση της επιλογής της πρώτης εικόνας στον πίνακα των υλικών.



Εικ. 3 Γενικές Πληροφορίες για κατασκευές από οπλισμένο σκυρόδεμα



Εικ.4 Γενικές Πληροφορίες για κατασκευές από χάλυβα

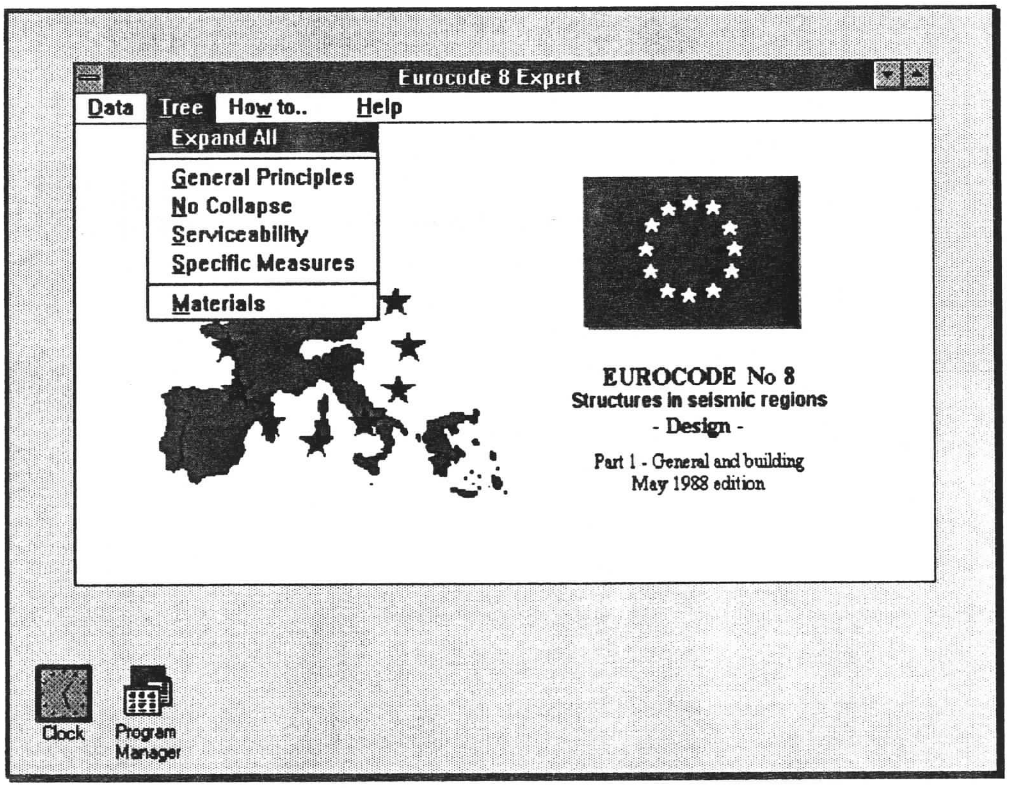


Εικ. 5. Γενικές πληροφορίες για κατασκευές απο τοιχοποιία

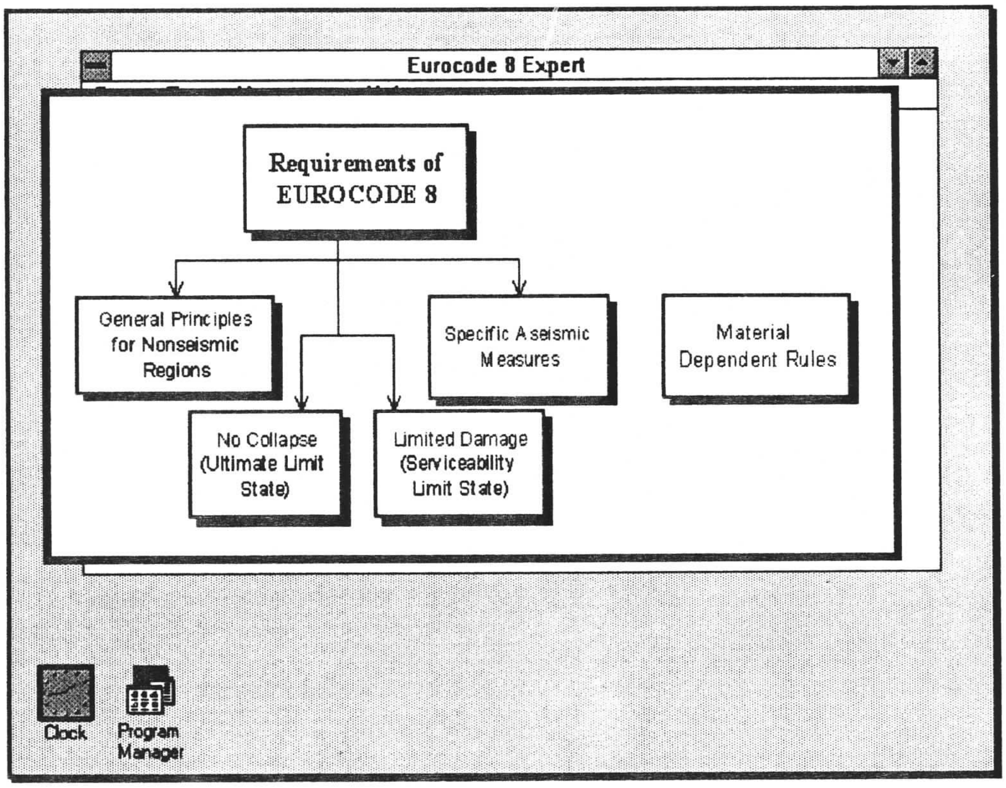
Σε όλες τις περιπτώσεις αναφέρονται και οι αντίστοιχοι Ευρωκώδικες που καλύπτουν τον πρωτεύοντα σχεδιασμό των επιμέρους κατασκευών. Οι πληροφορίες αυτές μπορούν να ενεργοποιήσουν αλγοριθμικά προγράμματα για επιμέρους θέματα.

Διερεύνηση της δομής των διατάξεων του Ευρωκώδικα No. 8.

Με το πάτημα του πλήκτρου Tree (Εικ. 6), το σύστημα ενεργοποιεί το έμπειρο σύστημα που παρέχει τη διαπλοκή των ενοτήτων του Ευρωκώδικα No. 8. Προτιμήθηκε η γραφική αναπαράσταση του δέντρου της διασύνδεσης των διαφόρων ενοτήτων του Ευρωκώδικα με την παρακάτω σύμβαση. Οι κόμβοι του δέντρου είναι δύο ειδών, κόμβοι που παρίστανται με ορθογώνια με μαύρη σκιά που σημαίνουν ότι με ένα click του ποντικιού επάνω τους το δέντρο ανοίγει προς τα κάτω, εκτός από τον πρώτο κόμβο που επιστρέφει πίσω. Οι κόμβοι χωρίς σκιά αποτελούν φύλλα του δέντρου και έτσι δεν ενεργοποιούνται. Η πρώτη οθόνη του δέντρου παρουσιάζεται στην Εικ. 7, όπου όλοι οι κόμβοι έχουν σκιά και άρα ενεργοποιούμενοι αποκαλύπτουν την παρακάτω δομή του δέντρου.

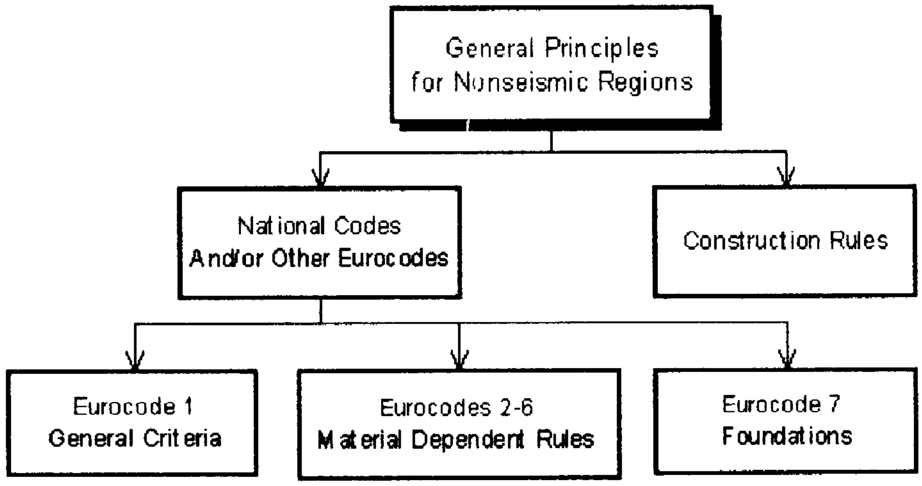


Εικ. 6. Ενεργοποίηση του δέντρου γνώσης

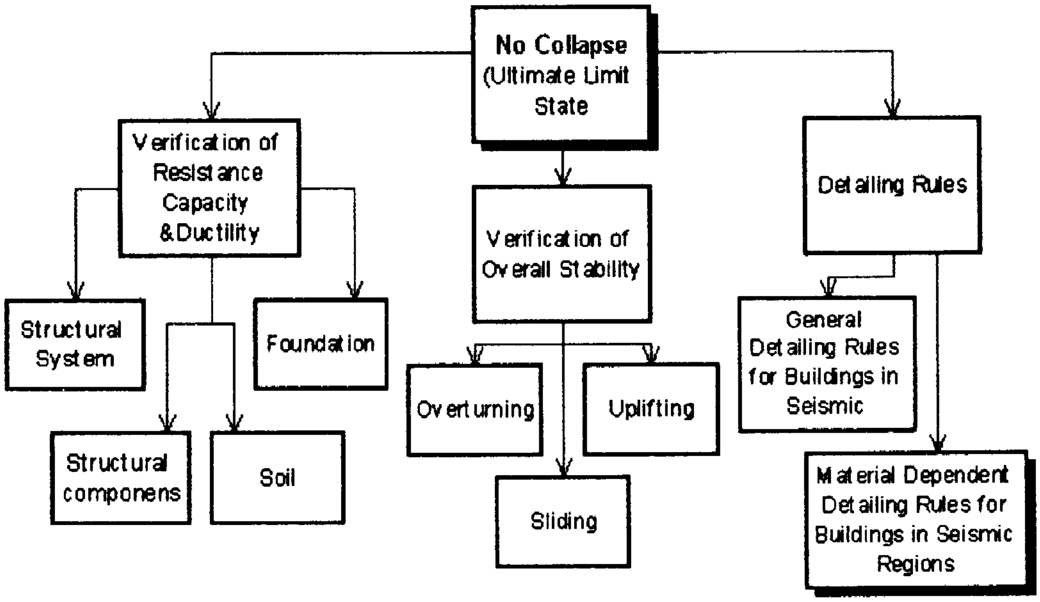


Εικ. 7 Η ρίζα της δενδροειδούς δομής του Ευρωκώδικα No. 8

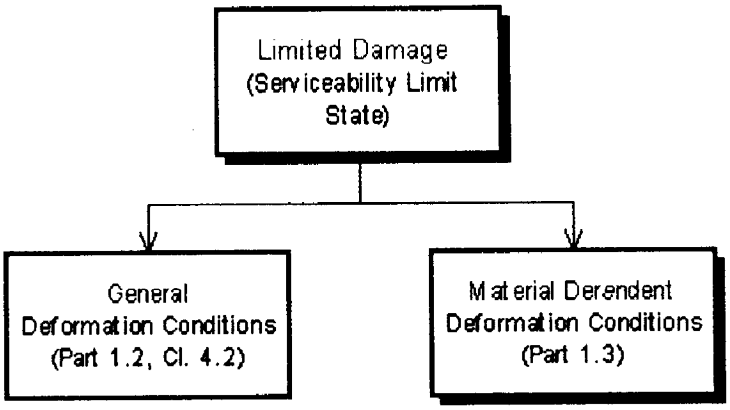
Σε κάθε τέτοια εικόνα πατώντας το πλήκτρο F1, που συνήθως χρησιμοποιείται για επεξηγήσεις, ένα κυλιόμενο παράθυρο ανοίγει από το οποίο ο χρήστης μπορεί να επιλέξει μία από τις ενότητες που εμφανίζονται στους κόμβους της συγκεκριμένης οθόνης και να βρεθεί στο αντίστοιχο κεφάλαιο του Ευρωκώδικα με τη μορφή κειμένου Hypertext και με τα χαρακτηριστικά που αναφέρονται σε παρακάτω ενότητα. Η οθόνη που προκύπτει με την ενεργοποίηση του πλήκτρου F1 για την πρώτη οθόνη του δέντρου παρουσιάζεται στην Εικ. 7. Η αναπαράσταση του δέντρου των διασυνδεδεμένων ενοτήτων του κώδικα εμφανίζεται στις παρακάτω εικόνες 8 έως 32.



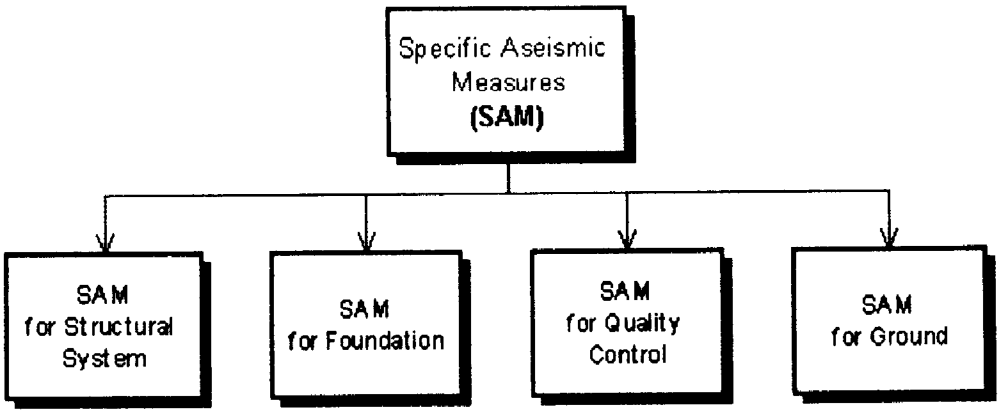
Εικ. 8. Γενικές Αρχές.



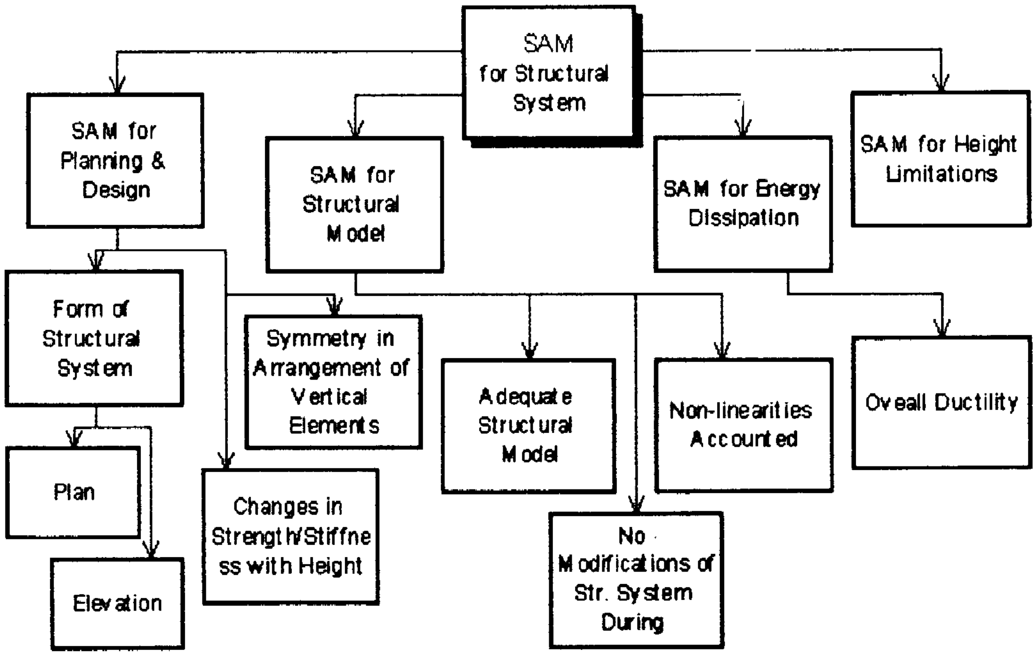
Εικ. 9. Αποφυγή κατάρευσης



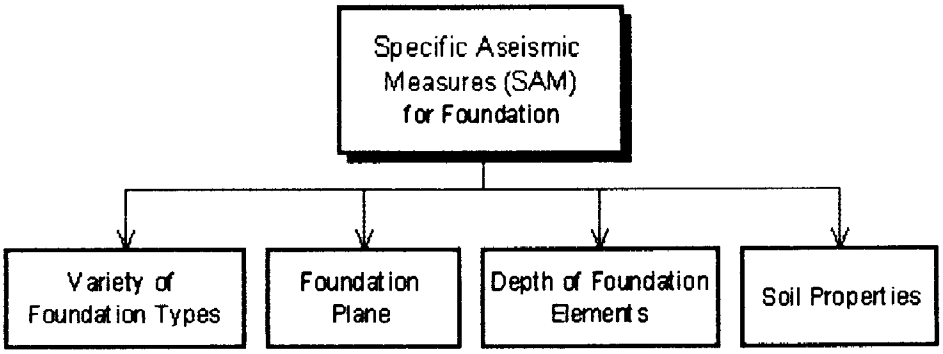
Εικ. 10. Οριακές καταστάσεις λειτουργικότητας



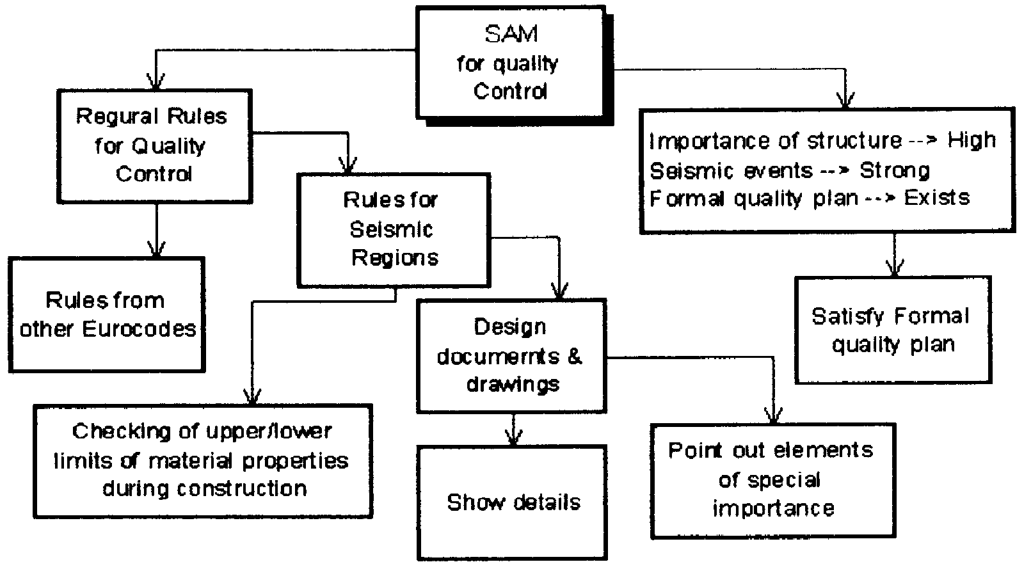
Εικ. 11. Ειδικά Αντισεισμικά μέτρα



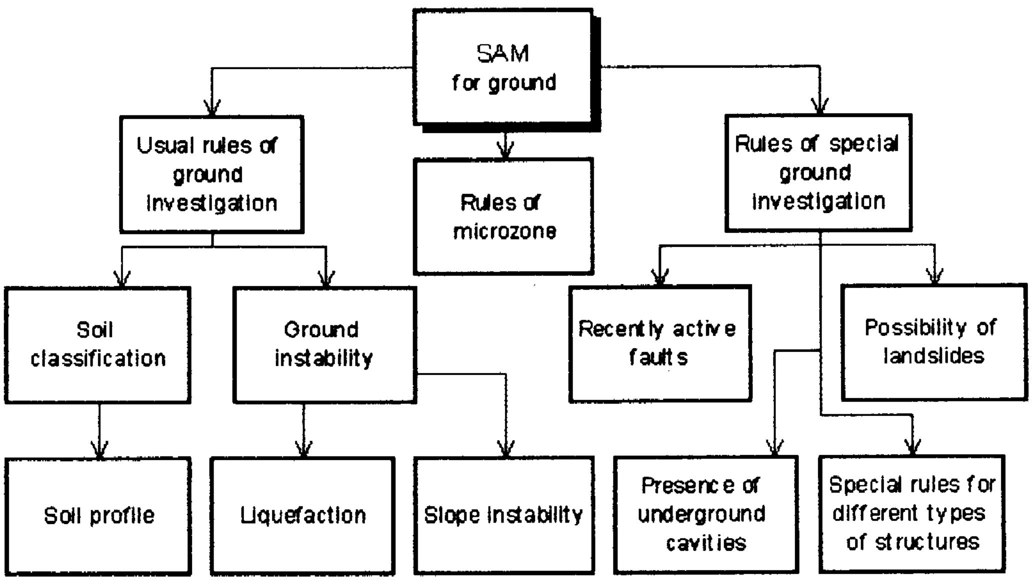
Εικ. 12. Ειδικά μέτρα για το δομικό σύστημα



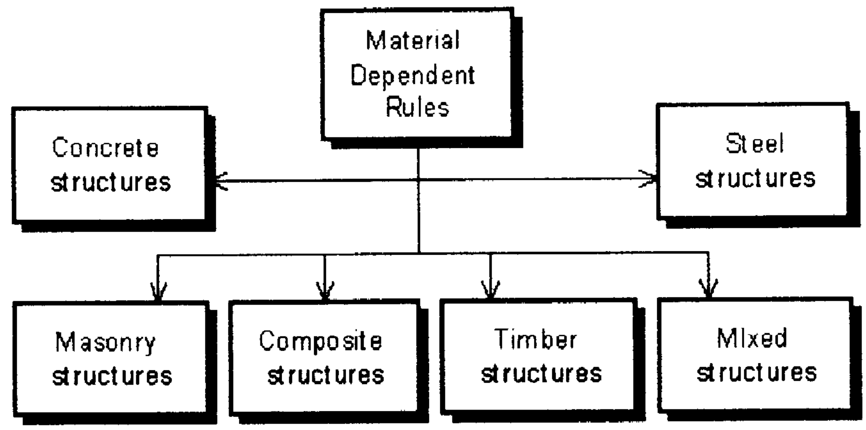
Εικ. 13. Ειδικά μέτρα για τη θεμελίωση



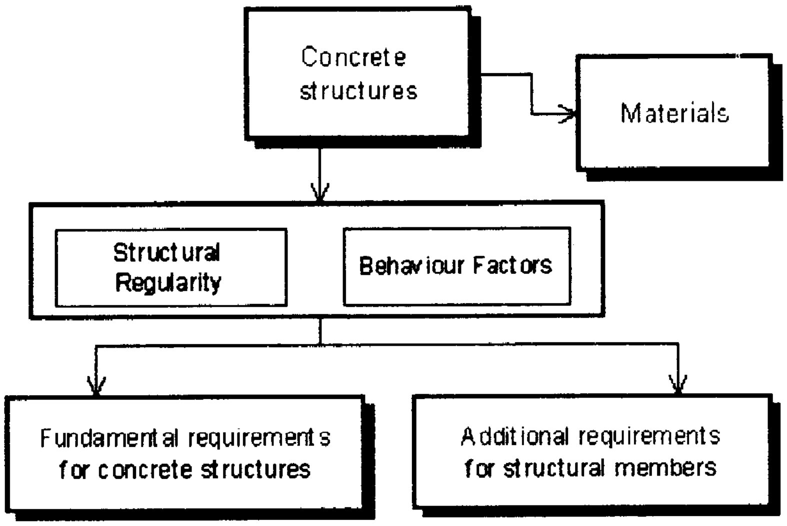
Εικ. 14. Ειδικά μέτρα για το ποιοτικό έλεγχο



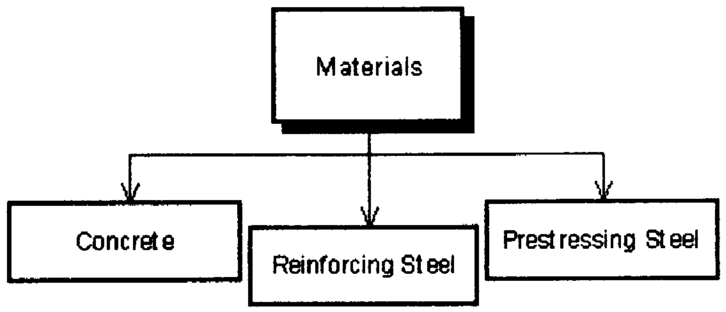
Εικ. 15. Ειδικά μέτρα για το έδαφος



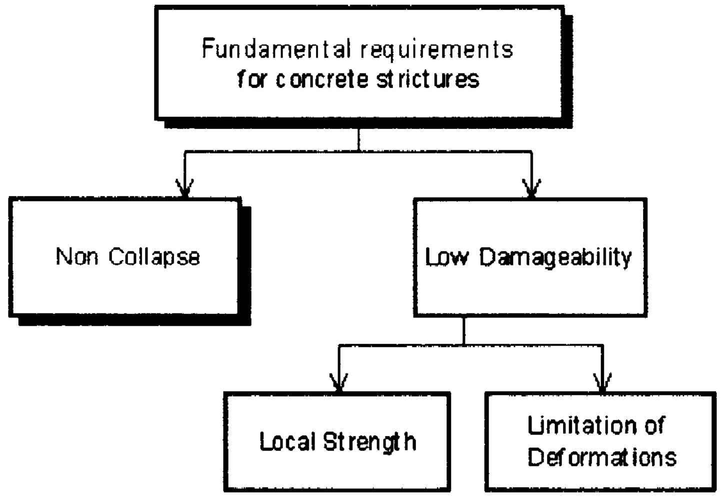
Εικ. 16. Κανόνες εξαρτώμενοι απο το υλικό



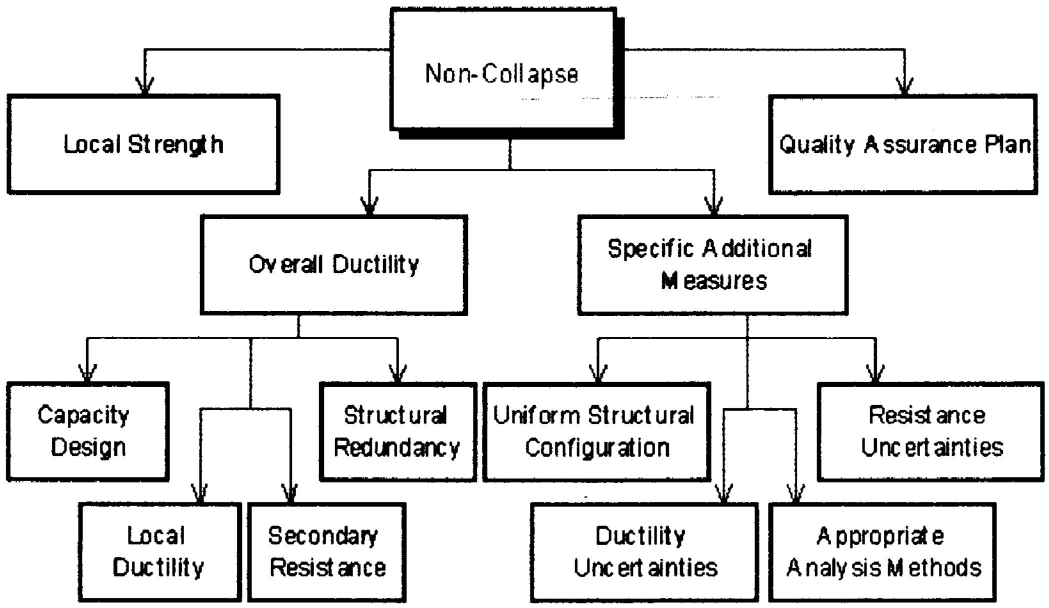
Εικ. 17. Κατασκευές σκυροδέματος



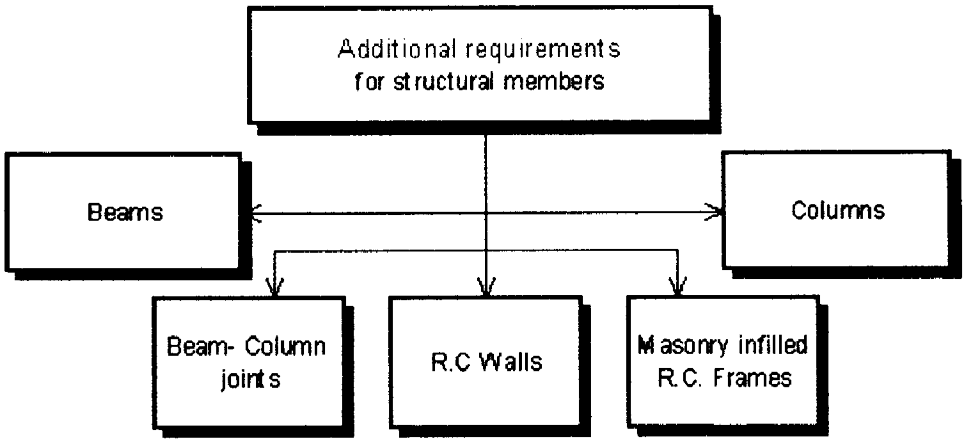
Εικ. 18. Υλικά



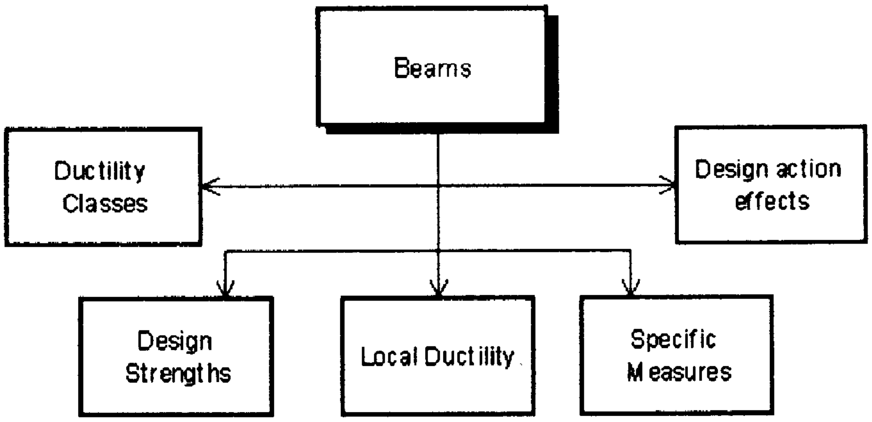
Εικ. 19. Βασικές απαιτήσεις



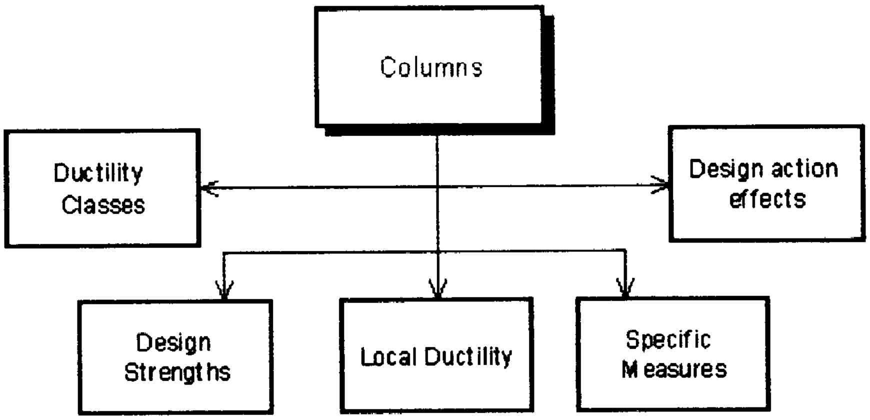
Εικ. 20. Οριακή κατάσταση αστοχίας



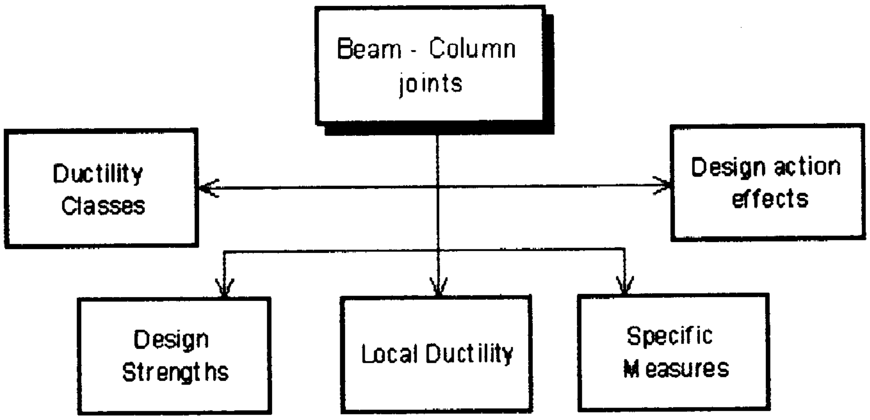
Εικ. 21. Πρόσθετες απαιτήσεις για τα μέλη της κατασκευής



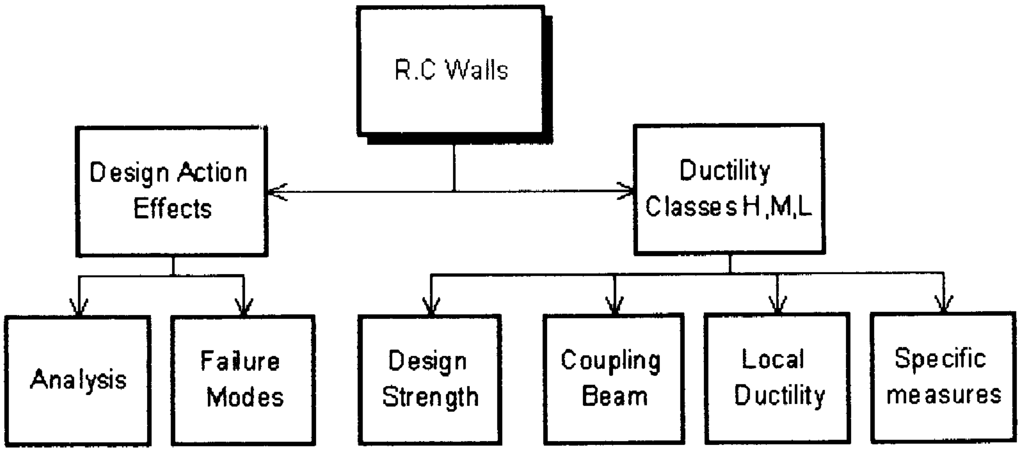
Εικ. 22. Δοκοί



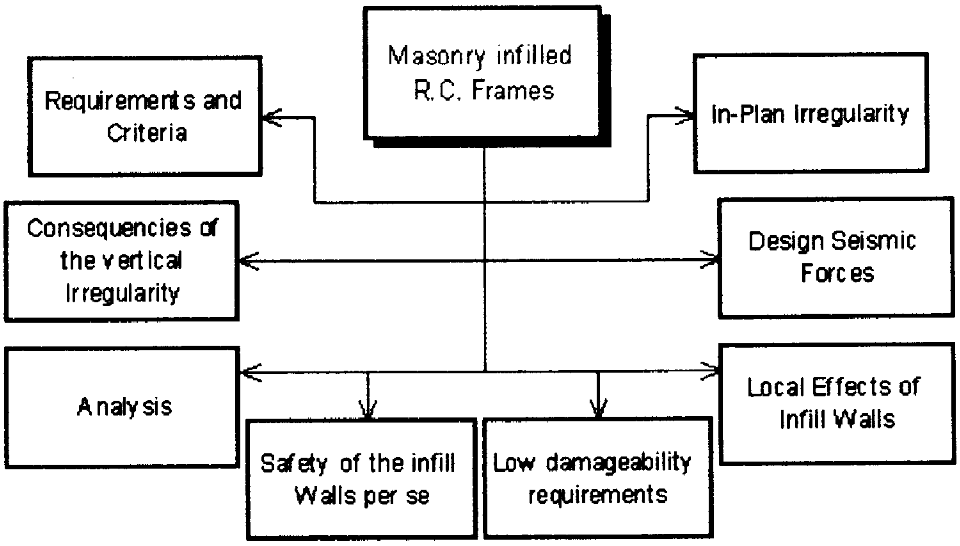
Εικ. 23. Υποστυλώματα



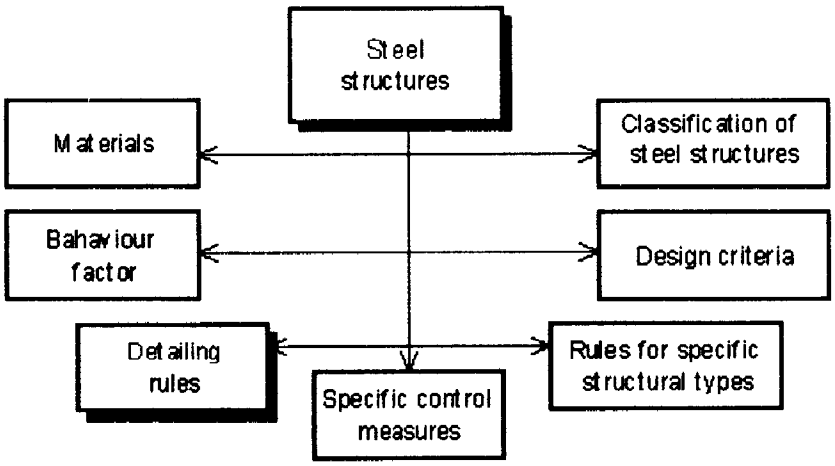
Εικ. 24. Κόμβος δοκού-υποστυλώματος



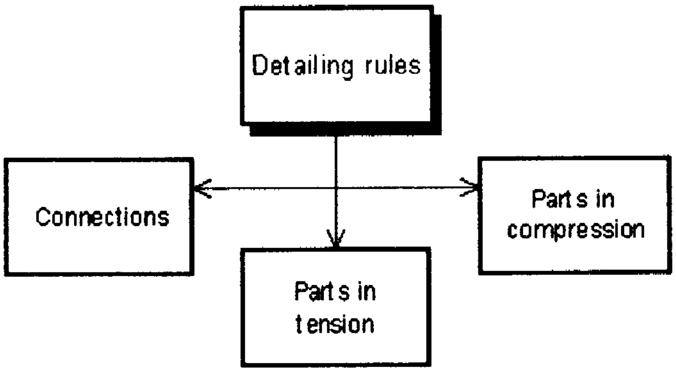
Εικ. 25. Τοιχώματα οπλισμένου σκυροδέματος



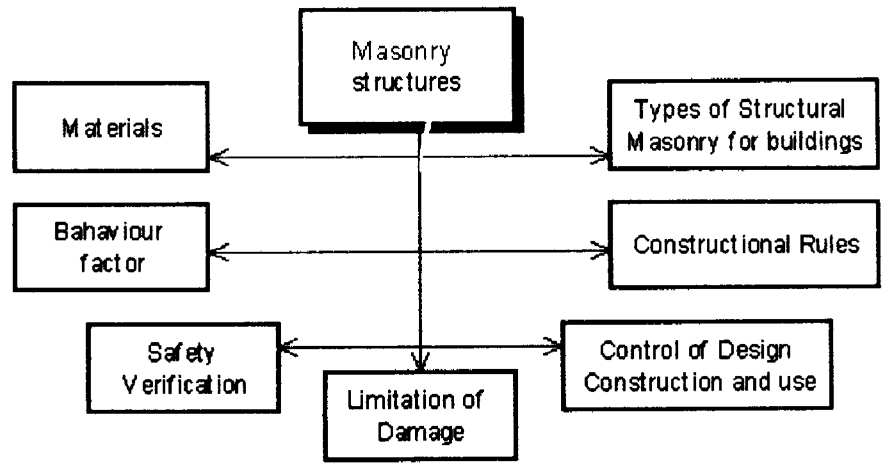
Εικ. 26. Τοιχοποιία πλήρωσης πλαισίων οπλισμένου σκυροδέματος



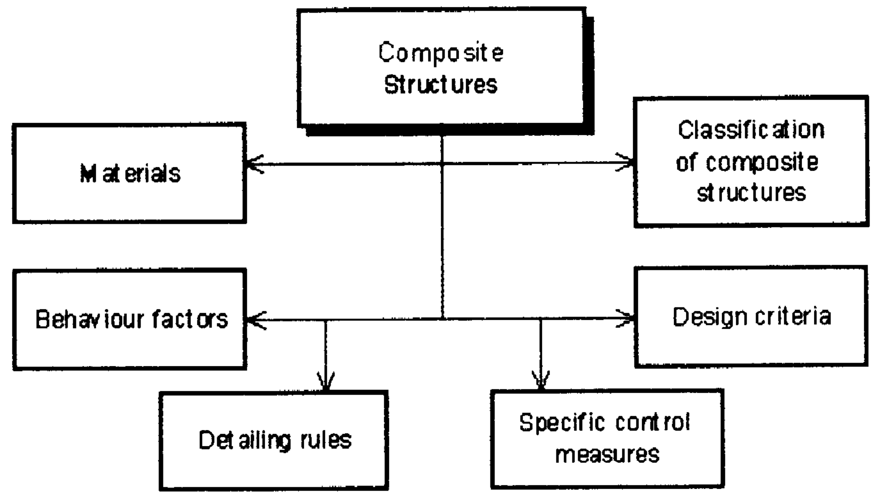
Εικ. 27. Κατασκευές από χάλυβα



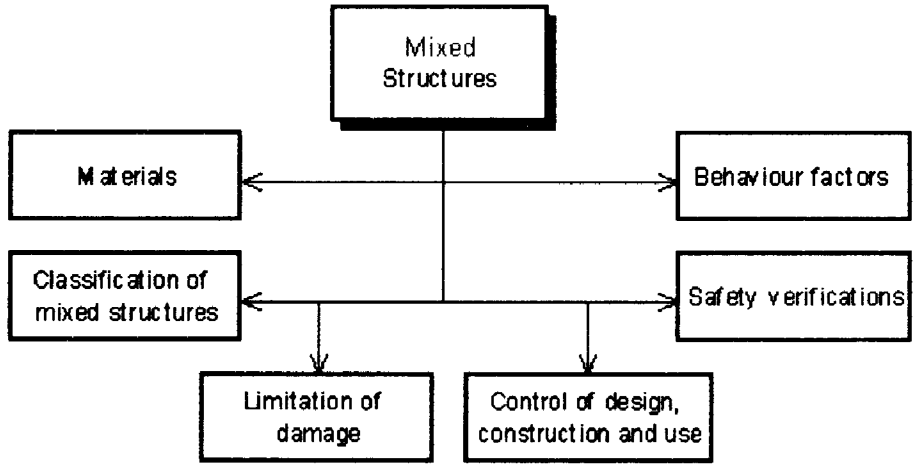
Εικ. 28. Κανόνες μόρφωσης λεπτομερειών



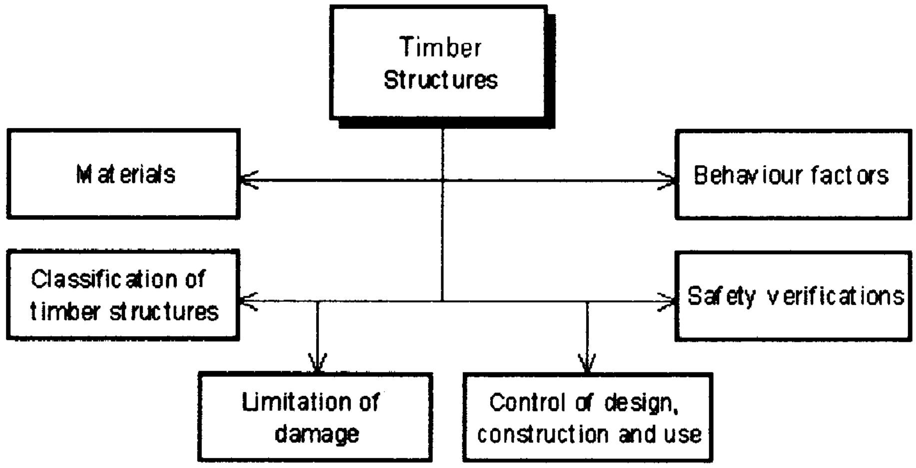
Εικ. 29. Κατασκευές απο τοιχοποιία



Εικ. 30. Συμμικτές κατασκευές



Εικ. 31. Μικτές κατασκευές

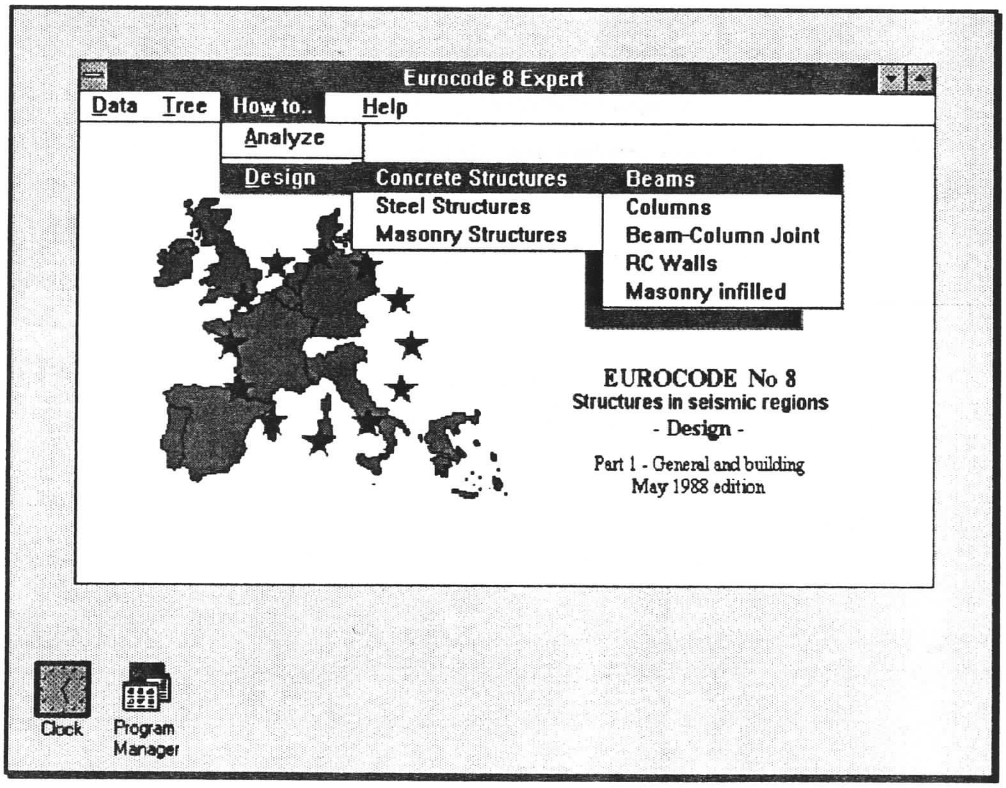


Εικ. 32. Ξυλινές κατασκευές

Το δέντρο αυτό αποτελεί τη βάση γνώσης του συστήματος. Συνήθως, στα περισσότερα έμπειρα συστήματα παρέχεται ως αποτέλεσμα της σύνδεσης μεμονωμένων κανόνων που εισάγονται στο σύστημα. Κάτι τέτοιο όμως προσφέρεται στις περιπτώσεις που η διαπλοκή των κανόνων είναι πολύ εκτεταμένη και δε διαπνέεται από μία απλή λογική όπως συμβαίνει σε ένα κώδικα. Έτσι κρίθηκε σκόπιμη η εξ αρχής προβολή της διαπλοκής του δέντρου στις βασικές του διακλαδώσεις και η αναφορά στις επιμέρους λεπτομερέστερες διατάξεις από ένα βάθος και κάτω. Η ιδιαιτερότητα των κωδίκων σε σχέση με τα τυπικά έμπειρα συστήματα θα λέγαμε ότι έγκειται στην πολυπλοκότητα των επιμέρους διατάξεων δηλ. στη δυσκολία να ανατρέξει κανείς και να συλλέξει τις διατάξεις που αφορούν το συγκεκριμένο θέμα, που τον απασχολεί, ενώ γνωρίζει τη γενική δομή του κώδικα.

How to ...

Με βάση το δέντρο της γνώσης που παρουσιάστηκε μπορούν να τεθούν διάφορα ερωτήματα-στόχοι που θα πρέπει το σύστημα να απαντήσει. Τα ερωτήματα αυτά διατυπώνονται με τη μορφή των ερωτημάτων "Πώς μπορώ να .." (how to ...) αναλύσω, ή να σχεδιάσω κάτι συγκεκριμένο. Η ενεργοποίηση του αντίστοιχου πλήκτρου στο menu μας οδηγεί σε επιμέρους menu όπως στην Εικ. 16 όπου ενεργοποιώντας το How to design columns of a concrete structure κάνω τις επιλογές που εμφανίζονται μαυρισμένες στα τρία ενεργοποιημένα menus.



Εικ. 33. Επιλογή στόχου

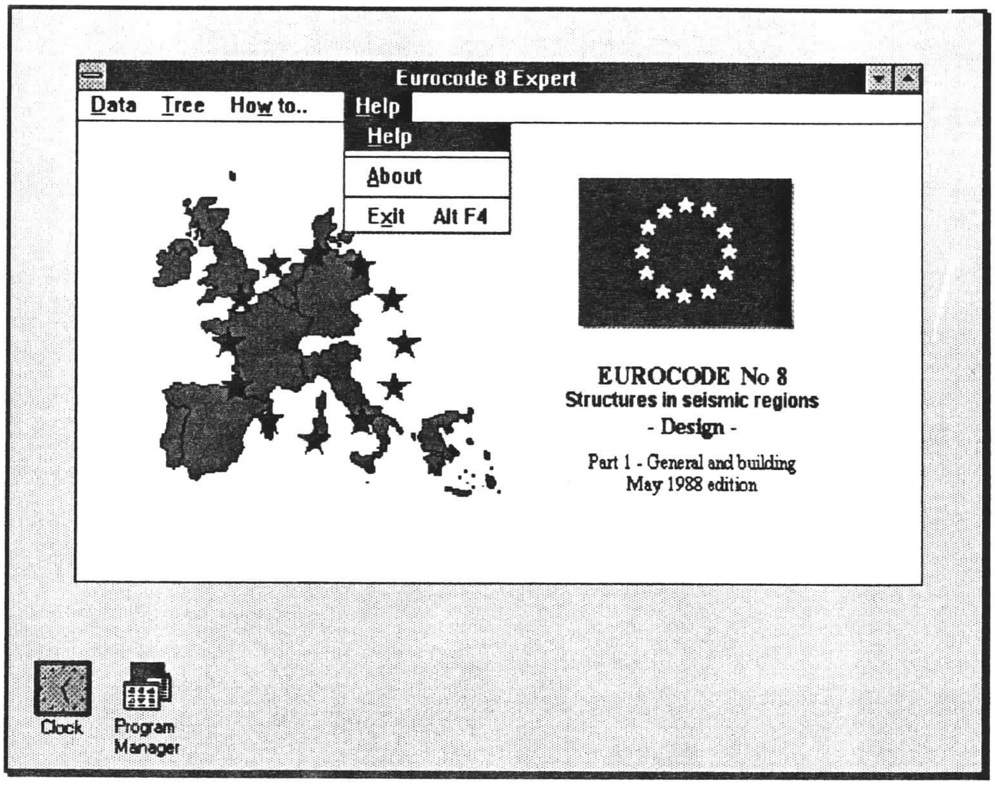
Με την επιλογή κάποιου συγκεκριμένου στόχου το σύστημα παρέχει τις αντίστοιχες διατάξεις του Ευρωκώδικα με τη μορφή hypertext μέσω του οποίου καλούνται και όλες οι απαιτούμενες διατάξεις που αποτελούν και την αποδεικτική οδό για την ανάλυση ή το σχεδιασμό κάποιας κατασκευής ή μέλους. Οι διατάξεις που εμπλέκονται μπορούν να απομονωθούν και να οδηγηθούν σε ένα αρχείο, το οποίο ο χρήστης μπορεί να τυπώσει για να το συμβουλεύεται.

Hypertext

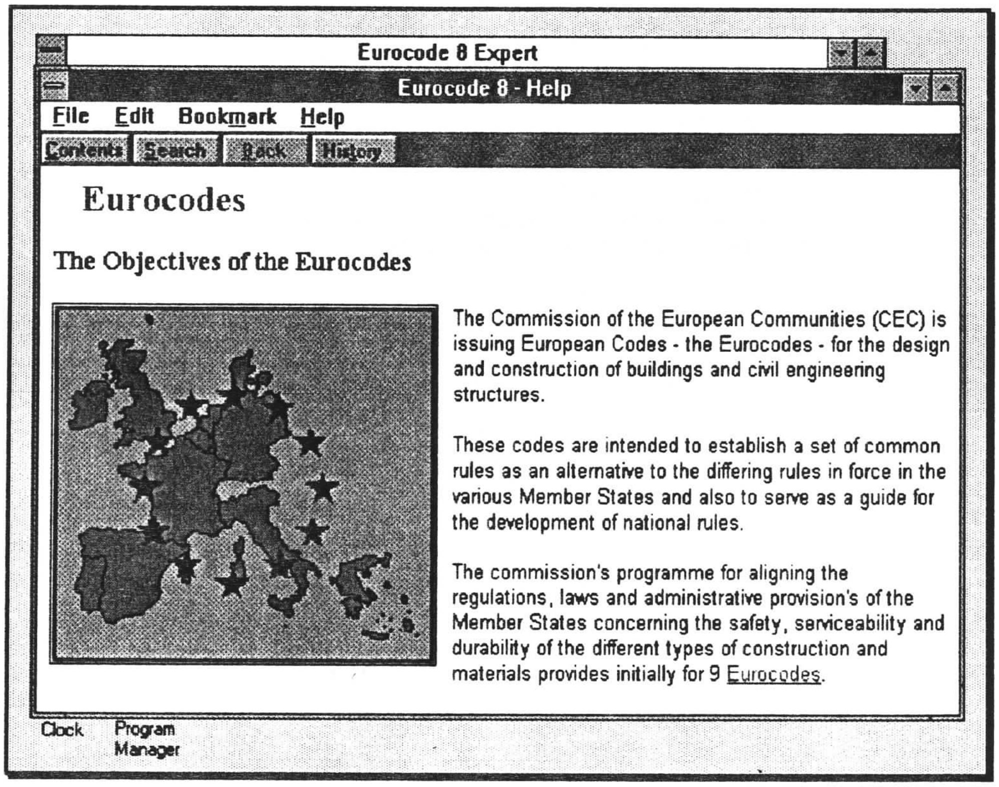
Βάση του όλου συστήματος αποτελεί η ανάπτυξη του συστήματος hypertext με τις διατάξεις του Ευρωκώδικα No. 8. Για το σκοπό αυτό ολόκληρος ο Ευρωκώδικας χρειάστηκε να αναπαραχθεί σε ηλεκτρονική μορφή, πληκτρολογώντας τα κείμενα του κώδικα και σχεδιάζοντας ηλεκτρονικά τα σχήματα που παρουσιάζονται στις διατάξεις του. Στη παρούσα μορφή δεν έχουν συμπεριληφθεί τα κείμενα και σχήματα που αφορούν στα σχόλια του κανονισμού, επειδή αναμένεται η τελική διατύπωση του κώδικα. Όπως αναφέρθηκε ήδη, το σύστημα hypertext αποτελεί εν γένει μία δομή δέντρου, οι κόμβοι του οποίου αποτελούνται από άλλα κείμενα hypertext. Έτσι η προσπέλαση στους διάφορους κόμβους του hypertext γίνεται με τρόπο αντίστοιχο με την αναζήτηση των προϋποθέσεων και συμπερασμάτων στους κανόνες ενός έμπειρου συστήματος για την απόδειξη κάποιου στόχου.

Το σύστημα hypertext αναπτύχθηκε χρησιμοποιώντας τις δυνατότητες των ρουτινών του SDK των MS-Windows 3.1. Η διαδικασία ανάπτυξης του συστήματος αυτού είναι επίπονη και απαιτεί ιδιαίτερες προγραμματιστικές ικανότητες, συνίσταται δε στον προσδιορισμό και κατάλληλη ηλεκτρονική καταγραφή των διασυνδέσεων επιμέρους ενοτήτων. Η εργασία αυτή πραγματοποιήθηκε κατά φάσεις και κατά ενότητα. Η διαπλοκή που έχει ενσωματωθεί στο παρόν σύστημα καλύπτει όλες τις διατάξεις του κώδικα πλην των σχολίων και με την αρχική σύνταξη του κώδικα. Στη φάση αυτή κρίθηκε σκόπιμο να μη γίνουν παραπομπές πέραν του κειμένου του κώδικα για να μην προσλάβει το σύστημα υποκειμενικό χαρακτήρα.

Το σύστημα hypertext ενεργοποιείται με το Help όπως εμφανίζεται στην Εικ. 34. Η μορφή του συστήματος hypertext είναι αυτή που εμφανίζεται στην Εικ.35. Το σύστημα hypertext διαθέτει το δικό του menu που περιλαμβάνει File, Edit, Bookmark, και Help πλήκτρα. Μετά τα menus του File και Edit,ο χρήστης μπορεί να ανοίξει ένα αρχείο και να κολλήσει το περιεχόμενο των οθονών που το Hypertext θα ανοίξει με τα διαδοχικά click στις υπογραμμισμένες λέξεις ή φράσεις κλειδιά, που αποτελούν τις προεπιλεγμένες λέξεις ή φράσεις, οι οποίες παραπέμπουν σε άλλα κείμενα hypertext. Με το πλήκτρο Bookmark ο χρήστης μπορεί να τοποθετεί σημεία ελέγχου ώστε να μπορεί να ανατρέξει σε αυτά. Με αυτό το τρόπο αλλά και με την ενεργοποίηση του annotate από το Edit ο χρήστης μπορεί να εισάγει τα δικά του σχόλια στα οποία μπορεί να ανατρέχει. Η λειτουργία αυτού του συστήματος ελέγχεται από τα τέσσερα πλήκτρα Contents Search Back και History. Με το πρώτο πλήκτρο ενεργοποιούνται τα περιεχόμενα του κώδικα, από όπου ο χρήστης μπορεί να επιλέξει το

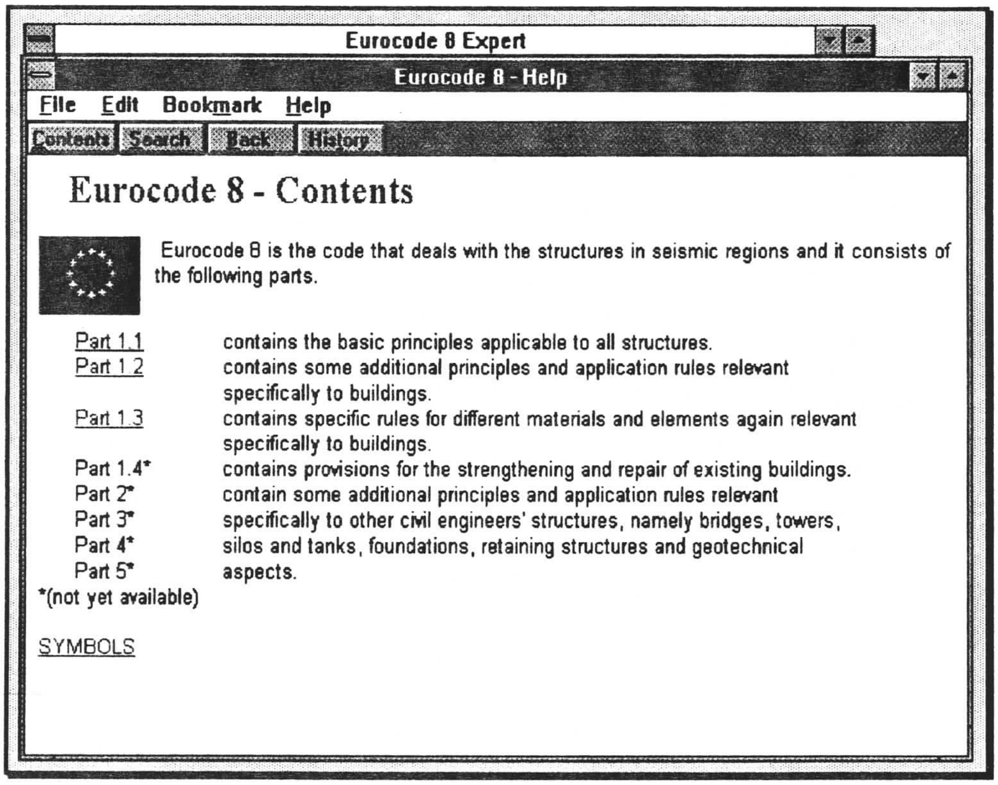


Εικ. 34. Ενεργοποίηση του Help



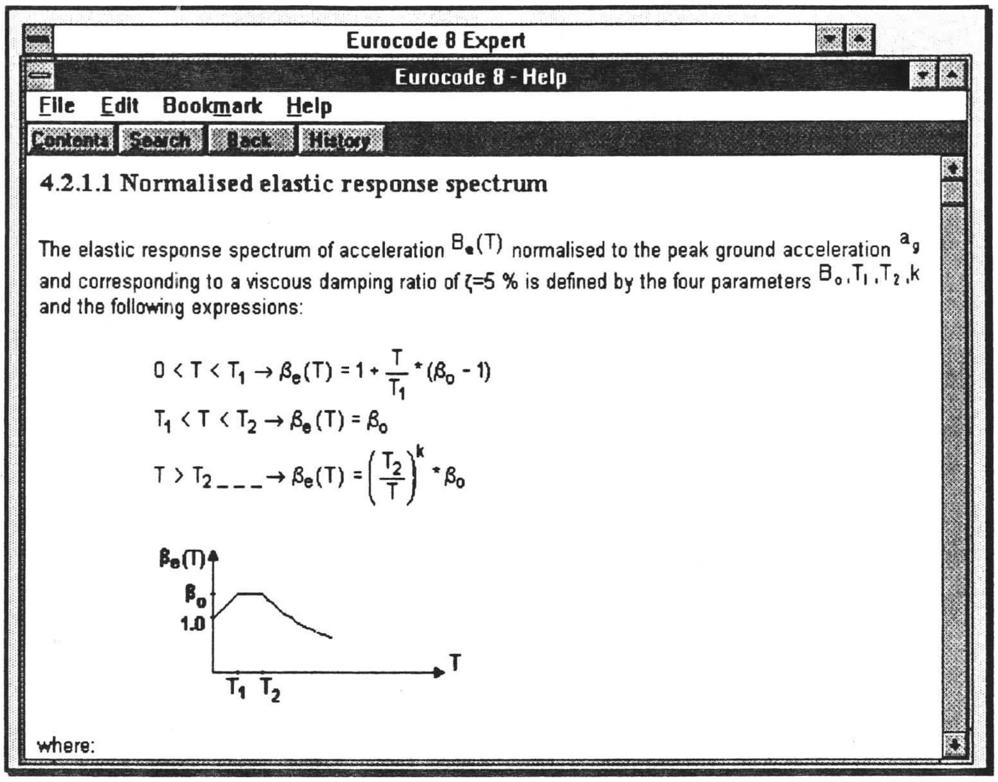
Εικ. 35. Αρχική οθόνη του συστήματος hypertext

θέμα που τον ενδιαφέρει το οποίο είναι και αυτό hypertext. Με το πλήκτρο Back ο χρήστης μπορεί να επιστρέψει στο προηγούμενο hypertext, ενώ με το πλήκτρο history είναι σε θέση να ανατρέξει στην ιστορία της διαδρομής που έχει ακολουθήσει και να ενεργοποιήσει αυτήν που επιθυμεί , χωρίς να ακολουθήσει τη βήμα προς βήμα πορεία επιστροφής χρησιμοποιώντας το πλήκτρο back διαδοχικά. Οι δυνατότητες αυτές υπάρχουν σε όλα τα συστήματα Help των MS-Windows και εξηγούνται στο Help των MS-Windows.Στην Εικ. 35 Η μόνη φράση που αποτελεί hypertext είναι η τελευταία λέξη Eurocodes. Η ενεργοποίηση του πλήκτρου Contents εμφανίζει την Εικ. 36.

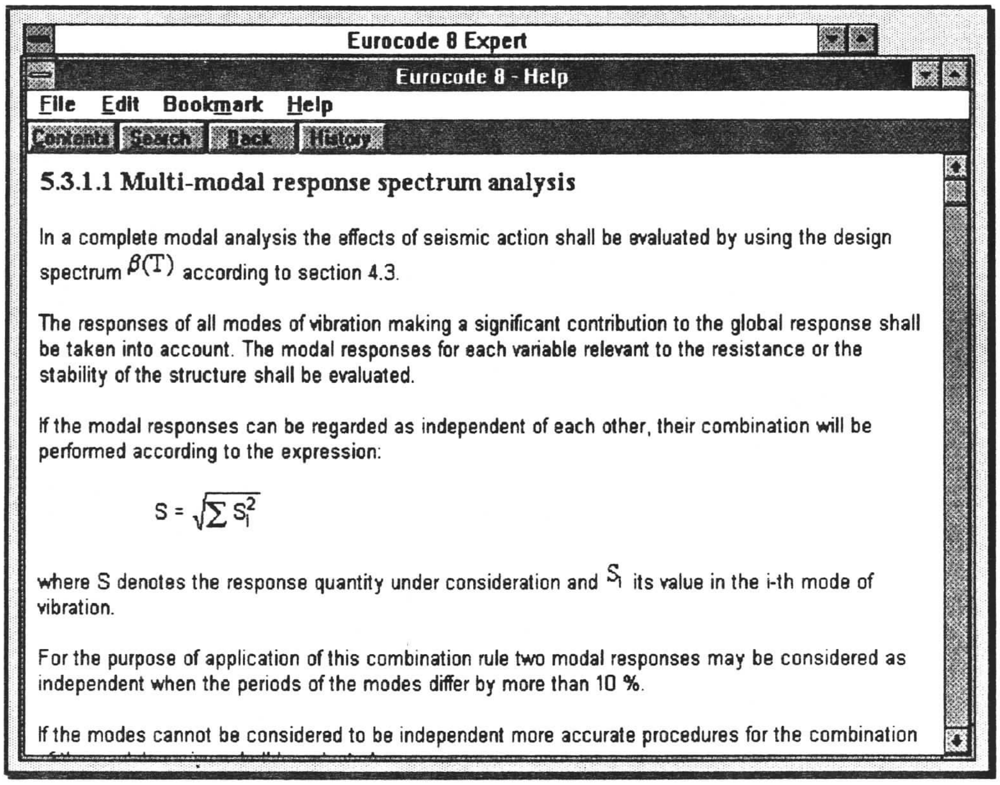


Εικ. 36. Τα περιεχόμενα του κώδικα

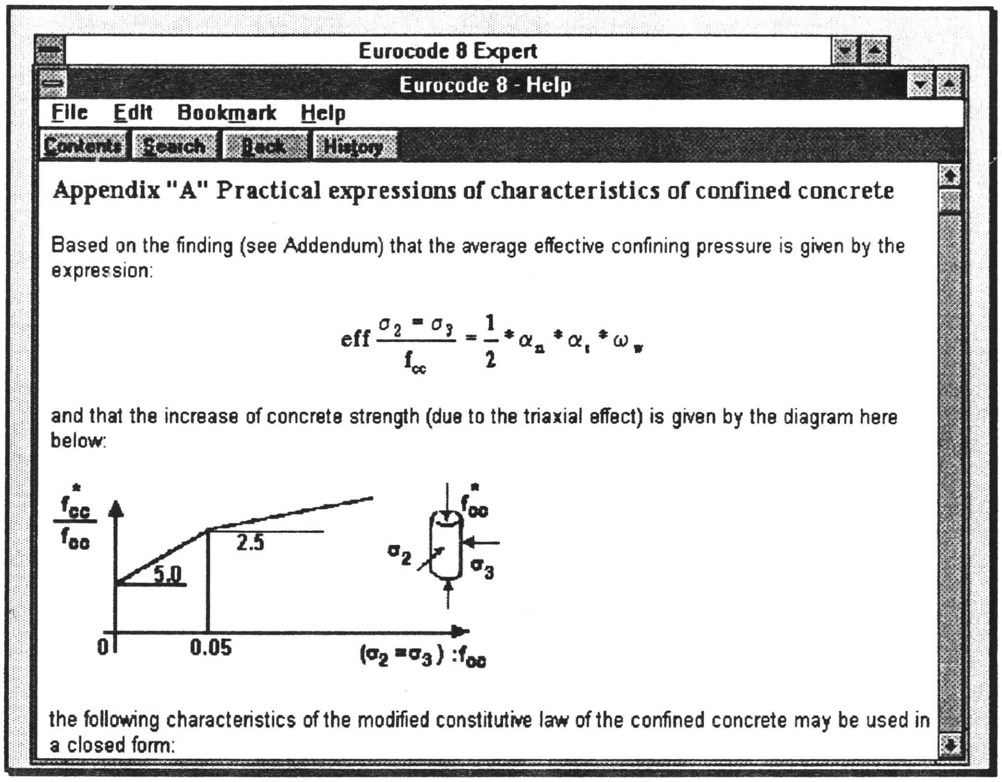
Στις Εικ. 37 εως 40 εμφανίζονται άλλες οθόνες που προέκυψαν από την ενεργοποίηση των εκφράσεων που περιγράφουν οι αντίστοιχοι τίτλοι. Συνολικά το πλήθος των οθονών είναι περίπου 250.



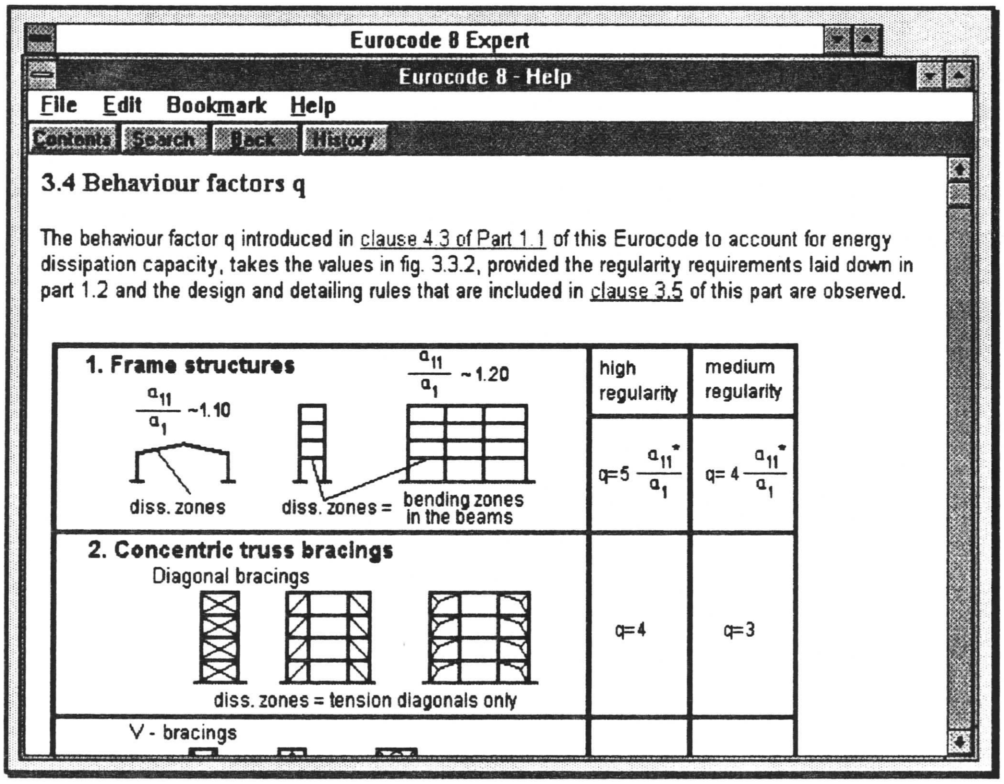
Εικ. 37. Ελαστικό φάσμα σχεδιασμού



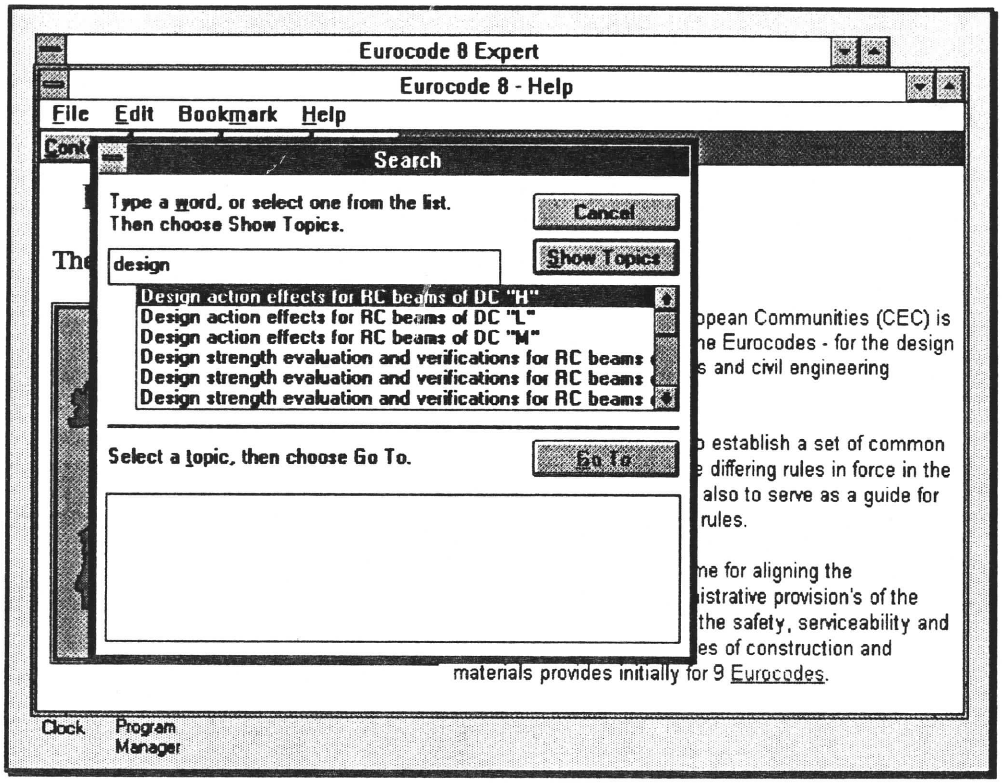
Εικ. 38. Φασματική ανάλυση



Εικ. 39. Χαρακτηριστικά περισφυγμένου σκυροδέματος



Εικ. 40. Συντελεστές συμπεριφοράς χαλύβδινης κατασκευής



Εικ. 41.. Λειτουργία αναζήτησης θέματος

Στην Εικ. 41 παρουσιάζεται η λειτουργία του πλήκτρου Search. Με την πληκτρολόγηση στην κατάλληλη περιοχή της ζητούμενης ενότητας, η λίστα στην κυλιόμενη οθόνη αυτόματα εντοπίζει αλφαβητικά τις αντίστοιχες διατάξεις. Επιλέγοντας μία συγκεκριμένη διάταξη το σύστημα ενεργοποιεί την αντίστοιχη διάταξη hypertext.

Απαιτήσεις σε hardware και Software

Το σύστημα για να τρέξει χρειάζεται έναν προσωπικό υπολογιστή PC τουλάχιστον 386 με μνήμη τουλάχιστον 2Mb και το περιβάλλον MS-Windows 3.x, συνιστώνται όμως 4 Mb.

Χρήση του συστήματος από ομάδες σύνταξης κανονισμών

Συμπληρωματικά με τις προτάσεις προς το Ευρωπαϊκό κέντρο επισημαίνεται ότι το σύστημα αυτό και παρόμοια του, μπορούν να αναπτύσσονται παράλληλα με τη διαδικασία σύνταξης των κωδίκων. Η οργάνωση της δουλειάς μπορεί να παρακολουθεί την οργάνωση της ανάπτυξης ενός κώδικα, με την προσθήκη μίας επιπλέον φάσης στο τέλος κάθε επιμέρους σταδίου. Κατά τη φάση αυτή οι ομάδες εργασίας θα πρέπει να παραδίνουν σε ηλεκτρονική μορφή τα κείμενα τους σε μία ομάδα που έχει την ευθύνη της ανάπτυξης του πληροφοριακού συστήματος. Στην ομάδα αυτή θα πρέπει να μετέχει και από ένας αντιπρόσωπος της κάθε ομάδας εργασίας ως σύνδεσμος που θα παρέχει εξηγήσεις για την απόδοση του έργου της ομάδας του. Το έργο της ομάδας ηλεκτρονικής τεκμηρίωσης του κανονισμού θα συνίσταται α) στην ανάπτυξη του συστήματος hypertext και τον προσδιορισμό των αλληλεξαρτήσεων των επιμέρους διατάξεων και β) στη σύνταξη των διαγραμμάτων ροής για τους ελέγχους που προβλέπει ο κανονισμός με όλη την αλληλουχία των υπολογισμών, κατά το πρότυπο του LRFD. Το προϊόν της σύνθεσης αυτής ανά φάση θα παραδίδεται στις ομάδες για σχολιασμό, εμπλουτισμό ή αναθεώρηση. Με αυτή την ανελικτική πορεία κρίνεται ότι θα διευκολυνθεί πολύ ο συντονισμός των ομάδων καθώς και ο εντοπισμός προβλημάτων στη ροή των υπολογισμών που επιβάλλει ένας κανονισμός για το σχεδιασμό ενός έργου. Θα συμβάλλει επίσης και στην ομογενοποίηση των κειμένων και διατάξεων του κανονισμού και γενικότερα στην απλοποίηση της διαδικασίας σύνταξης ενός κανονισμού.

Πρόταση Επέκτασης του Έργου

Η μορφή του Ευρωκώδικα Νο. 8 που χρησιμοποιήθηκε ως βάση ανάπτυξης του συστήματος δεν είναι η οριστική. Μόλις ο Ευρωκώδικας Νο. 8 οριστικοποιηθεί, προτείνεται η επέκταση του έργου ώστε να περιλάβει τις αλλαγές και τα σχόλια στην τελική τους μορφή.

Επιπλέον μπορεί να αναπτυχθεί μία έκδοση του προγράμματος που συγκεκριμενοποιεί τις διατάξεις του Ευρωκώδικα Νο. 8 με τιμές μεγεθών που αφορούν την Ελλάδα σε συνδυασμό με το Νέο Αντισεισμικό Κανονισμό (Ν.Ε.Α.Κ.). Σ' αυτή τη περίπτωση είναι δυνατή και η κάλυψη των αλγοριθμικών τμημάτων του κανονισμού.

Τέλος κρίνεται σκόπιμο να πραγματοποιηθεί μία γενικότερη προσπάθεια για την απόδοση όλων των Ευρωκωδίκων τουλάχιστον με τη μορφή Hypertext και κατ' αρχήν στο επίπεδο της νοηματικής διασύνδεσης των διατάξεων και στη συνέχεια στο επίπεδο των αλγορίθμων. Η επιλογή του φορέα και η οργάνωση του νέου αυτού ερευνητικού έργου μπορεί να αποτελέσει πρόταση του Ευρωπαϊκού Κέντρου για το επόμενο πρόγραμμα.

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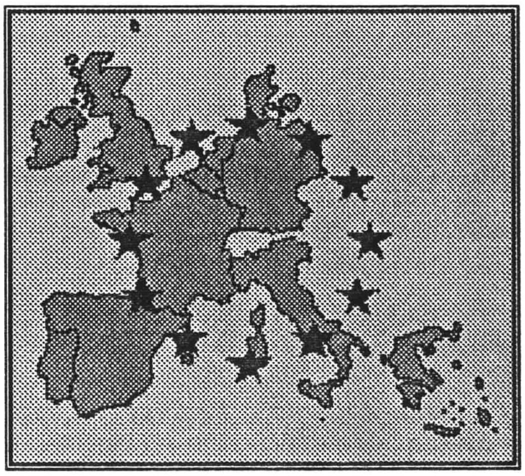
Eurocodes

The Objectives of the Eurocodes



The Commission of the European Communities (CEC) is issuing European Codes - the Eurocodes - for the design and construction of buildings and civil engineering structures.

These codes are intended to establish a set of common rules as an alternative to the differing rules in force in the various Member States and also to serve as a guide for the development of national rules.

The commission's programme for aligning the regulations, laws and administrative provision's of the Member States concerning the safety, serviceability and durability of the different types of construction and materials provides initially for 9 Eurocodes.

Eurocodes

Eurocode 1	with common unified rules for different types of construction and material
Eurocode 2	for concrete structures
Eurocode 3	for steel structures
Eurocode 4	for composite steel and concrete structures
Eurocode 5	for timber structures
Eurocode 6	for masonry structures
Eurocode 7	for foundations
Eurocode 8	for structures in seismic regions
Eurocode ...	for actions on structures

Eurocode 1

Eurocode 1 contains common unified rules for different types of construction and material

Eurocode 2

Eurocode 2 contains common unified rules for concrete structures

Eurocode 3

Eurocode 3 contains common unified rules for steel structures

Eurocode 4

Eurocode 4 contains common unified rules for composite steel and concrete structures

Eurocode 5

Eurocode 5 contains common unified rules for timber structures

Eurocode 6

Eurocode 6 contains common unified rules for masonry structures

Eurocode 7

Eurocode 7 contains common unified rules for foundations

Eurocode ...

Eurocode ... contains common unified rules for actions on structures

Eurocode 8 - Contents

Eurocode 8 is the code that deals with the structures in seismic regions and it consists of the following parts.

Part 1.1	contains the basic principles applicable to all structures.
Part 1.2	contains some additional principles and application rules relevant specifically to buildings.
Part 1.3	contains specific rules for different materials and elements again relevant specifically to buildings.
Part 1.4*	contains provisions for the strengthening and repair of existing buildings.
Part 2* Part 3* Part 4* Part 5*	contain some additional principles and application rules relevant specifically to other civil engineers' structures, namely bridges, towers, silos and tanks, foundations, retaining structures and geotechnical aspects.

*(not yet available)

Part 1.1 - 1. Introduction

Part 1.1 - 2. Requirements for structures and criteria for ensuring compliance with these requirements

Part 1.1 - 3. Special aseismic considerations regarding the ground

Part 1.1 - 4. Seismic action

Part 1.3 - 3. Specific rules for steel structures

Part 1.3 - 7. Specific rules for mixed structures

Part 1.3 - 8. Specific rules for elements

8.4. Limitation of damage

8.5. Control of design, construction and use

Symbols

Particular material-independent symbols used in this Eurocode are as follows:

B	total horizontal dimension of a building, parallel to the direction of the seismic action considered
E	seismic action in general, total seismic force
E_X, E_Y, E_Z	seismic action components
F_a	seismic force acting on an appendage supported by the main structure
F_i	horizontal seismic forces acting on weights W in static or simplified dynamic analyses
G	permanent actions
L	total horizontal dimension of a building, perpendicular to the direction of the seismic action considered
P	prestressing, action due to prestress
P_{tot}	total gravity load action on a storey level
Q_{ik}	variable loads at their characteristic values
R_d	design resistance capacity
S	response quantity in multi-modal response spectrum analyses
S_d	design action effect
S_i	response quantity in the i-th mode of vibration
T	fun fundamental vibration period of a linear - elastic single - degree of freedom system
T_1, T_2	limit T-values of the constant spectral acceleration branch in the normalized elastic or design response spectrum
V_{tot}	total seismic design shear action acting across the storey-considered
W_a	weight of an appendage supported by the main structure
W_i	weights of the masses considered in simplified dynamic or static analyses
a	spectral acceleration
a_g	peak ground acceleration
d	absolute maximum ground displacement Cement for a given soil profile under a given earthquake
d_e	displacement of a point of the structural system determined by linear analysis based on the design spectrum
d_j	width of a seismic structural joint
d_o	absolute ground displacement for a given soil profile corresponding to $a_g = g$
d_r	design inter storey drift (relative horizontal displacement of consecutive storey diaphragms)
d_s	displacement of a point of the structural system induced by the design seismic action

e_o	actual geometric eccentricity (distance between the stiffness Centre and a straight line through the mass centre running parallel to the seismic action considered
e_1	additional eccentricity taking into account the dynamic effect of simultaneous translational and torsional vibrations
e_2	accidental eccentricity of masses from their nominal location
g	acceleration of gravity
h	storey height
i	storey number; order of vibrational mode; slope of ground surface
k	exponent
l_s	substitute length in calculating additional eccentricities
n	total number of lumped masses considered in simplified dynamic analysis
q	behaviour factor
r	ratio between torsional and translational stiffness
si	horizontal deflections of points of the structure where weights W_i are assumed to be concentrated, corresponding to a rational a approximation for the fundamental mode shape of the structure
x	abscissa, - distance, in torsional calculations, of the element under consideration from the floor centre of gravity, measured perpendicularly to the direction of the seismic action
zi	height of the i-th mass point from the level of seismic excitation in simplified dynamic analysis
α	ratio of peak ground acceleration to the acceleration of gravity $\alpha = a_g : g$
$\beta(T)$	design spectrum of acceleration
$\beta_e(T)$	elastic spectrum of acceleration corresponding to a damping ratio 5%) normalized to the peak ground acceleration
bo	maximum $\beta_e(T)$ value
$\beta_s(T)$	site dependent value of $\beta_e(T)$
γ_i	importance factor
e_o	factor for determining horizontal seismic forces in static analysis
ζ	viscous damping ratio (damping normalized to the critical value)
n	correction factor for damping ratios ζ different from 5 %.
θ	stability coefficient for checking 2nd order effects
λ, μ	combination factors of seismic action components
v	factor that takes into account the difference in deformations due to the earthquakes with different probability of occurrence which are considered in the ultimate and serviceability limit state
ξ	correction factor in simplified torsional calculations
φ	ratio between combination factors ψ_{Ei} and ψ_{2i}
ψ_{Ei}	combination factor of O_{ik} for determining the inertia forces
ψ_{2i}	combination factor of Q_{ik} for verification purposes

Part 1.1

SEISMIC ACTIONS AND GENERAL REQUIREMENTS AND RULES FOR DESIGN

1.1 Object

This Eurocode provides the basis for the design and construction of structures in seismic regions and proposes operational rules for application. Its purpose is to ensure, with adequate reliability, that in the event of earthquakes

i) Human lives are protected

ii) Damages are limited

iii) Critical facilities remain operational.

The Principles which specify the requirements and criteria to be fulfilled by the structure to comply with this Eurocode are included in the right hand pages and denoted in the text by a vertical line in the margin. In this context Principles comprise both general statements and definitions for which there is no alternative as well as requirements and models for which no alternative is permitted unless specifically stated.

The Rules for Application are intended to be an acceptable method of satisfying the principles set out in the Eurocode but do not preclude the use of other operational rules which can be shown to satisfy these same principles. They are also included in the right hand pages of the Code.

1.2 Scope

This Eurocode is concerned with buildings and normal civil engineering structures.

This Part	contains the basic principles applicable to all these structures.
Part 1.2	contains some additional principles and application rules relevant specifically to buildings.
Part 1.3	contains specific rules for different materials and elements again relevant specifically to buildings.
Part 1.4	contains provisions for the strengthening and repair of existing buildings.
Parts 2, 3, 4 and 5	(not yet available) contain some additional principles and application rules relevant specifically to other civil engineers' structures, namely bridges, towers, silos and tanks, foundations, retaining structures and geotechnical aspects.

Special Structures associated with increased risks for the population such as nuclear power plants, large dams, structures for the processing or storage of particularly dangerous materials etc. are outside the scope of this Code.

The level of protection adequate for such facilities shall be established through proper criteria by the Competent National Authorities, on the basis of the consequences of failure specific to each facility. For many of such facilities, additional safety requirements, criteria and application rules will have to be introduced, related to the functioning of the internal subsystems comprised within the structure.

Not included within the scope of this Code are also structures where special measure's are taken to isolate them (fully or partially) from the effects of earthquake motions, albeit the fact that some of the principles contained in this Code may be applicable in these cases as well.

1.3 Assumptions

For the use of the present Eurocode, the following basic assumptions, established in Eurocode 1 apply:

i) Projects are designed by appropriately qualified and experienced personnel.

ii) Adequate supervision is always provided in factories and on site.

iii) Construction is carried out by personnel having the required skill and experience.

iv) The structure is adequately maintained.

v) The intended use of the structure will not be changed for the worse unless re-calculation is carried out.

1.4 Units

As in other Eurocodes S.I. Units in accordance with the ISO Standard 4357 "Rules for the use of international systems of units (S.I.) in buildings" are used.

1.5 Symbols

The symbols used comply with the ISO Standard 3898 "Bases for the Design of Structures-Notations-General Symbols".

For the material-dependent symbols as well as for symbols not specifically connected to earthquakes the provisions of the other Eurocodes apply.

1.6 Reference Codes

For the application of this Eurocode reference shall be made to the other relevant Eurocodes in so far as this code is not self sufficient but contains only those provisions that, in addition to the provisions of the relevant Eurocodes, must be observed for structures in seismic regions.

2.1 Requirements

For the planning, design and construction of structures in seismic regions, in addition to the general principles applicable in non seismic regions, the requirements of Clause 2.1.1 and Clause 2.1.2 shall be met with an appropriate degree of reliability.

Target reliability's shall be established on the basis of the consequences of failure considering both aspects of safety and serviceability. Consequences of failure, in which monetary and non monetary losses are included, depend principally on the use of the structures, on their contents and on the importance of their functions.

Different levels of reliability are thus envisaged. This reliability differentiation is obtained by amplifying the seismic action effects through a factor γ_I called "importance factor" (Clause 6.2).

1. No collapse requirement (Ultimate limit state)
2. Limiting susceptibility to damage (Serviceability limit state)

2.1.1 No collapse requirement (Ultimate limit state)

The entire structure shall be planned, designed and constructed such that it can withstand without local or general collapse the design seismic action defined in chapter 4, thus retaining its integrity and a residual capacity after this seismic action has ceased.

2.1.2 Limiting susceptibility to damage (Serviceability limit state)

The structure as a whole, including structural and non-structural elements shall be planned, designed and constructed such that it is protected against the occurrence of damages and limitations of use as a consequence of an earthquake having a larger probability of occurrence than the one to be considered in Clause 2.1.1.

2.2 Criteria

Design criteria comprise the set of operations to be performed in order to satisfy the general requirements set forth in Clause 2.1.

These operations include checking both the ultimate and the serviceability limit states as well as specific measures indicated as follows:

- i) Criteria concerning reliability against collapse
- ii) Criteria for limitation of damage
- iii) Specific Aseismic measures

2.2.1 Criteria concerning reliability against collapse

An adequate degree of reliability against collapse is ensured if the detailing rules given in parts 1.2, 1.3 and 2-5 are observed and the verifications listed below performed. The action effects to be used for these verifications shall be calculated in accordance with chapters 4 to 6 including, if applicable the capacity design procedures set forth in part 1.3 etc.

1) Verification of resistance capacity and ductility

   The structural system as a whole as well as any of its components shall be verified as having adequate strength and ductility to withstand the actions referred above.

   In the strength verifications of structural elements the possible influence of 2nd order effects on the action effects shall be taken into account. It shall be verified that both the foundation and the foundation soil are able to resist the action effects transmitted by the superstructure without appreciable residual deformations. In determining the actions transmitted to the soil, due consideration shall be given to the actual maximum resistance capacity of the structural element transmitting the action.

   The capability of a structural system to resist seismic actions in the post-elastic range is allowed for by means of the behaviour factor q. This parameter takes into account the energy dissipation capacity through ductile behaviour.

2) Verification of overall stability

   Verification of stability shall be performed for the structure as a whole considering the actions referred above. Herein are included the verifications of overturning, sliding and uplifting.

2.2.2 Criteria for limitation of damage

An adequate degree of reliability against unacceptable damage is ensured if the deformation conditions defined in the parts of the Code relevant to the different structures and material types are satisfied (see part's 1.2, 1.3, 2.5).

The action effects to be used shall be calculated in accordance with chapters 4 to 6 of the present part.

2.2.3 Specific aseismic measures

The uncertainties inherent in aseismic design and their consequences shall be minimised by taking the following measures:

1) Planning and design

a) As far as practicable, structures shall have simple form both in plan and elevation.

In addition a symmetrical arrangement of elements to minimise torsional action should be sought and abrupt changes in strength with height should be avoided. In order to select forms and procedures to be used in the design, the attribute of levels of structural regularity is defined for the different types of structures in parts 1.2, 1.3 etc. of this Code.

(b) The analysis shall be based on an adequate structural model, which, when appropriate, shall take into account the influence of non structural members.

No change, modifying the structural model used or the assumptions made, is allowed during the constructions' phase or the subsequent life of the structure, unless proper justification and verification is provided, including the check that no undesirable over strengths are introduced.

(c) The structure shall be designed and proportioned to have sufficient overall ductility to ensure adequate energy dissipation.

2) Height and other limitations

In order to limit the consequences of damage, National Authorities may specify restrictions on the height or other characteristics of a structure depending on local seismicity, soil conditions, city planning and environmental planning.

3) Foundations

The use of different foundation types for the same structure shall be avoided, unless shown to be acceptable through appropriate analysis. The structures should be founded on a horizontal plane, with the foundation elements having approximately the same depth and on soils without significantly different properties.

Measures should be taken to cope with relative displacements of individual foundations.

4) Quality plan

National Authorities may, in special cases of seismicity and importance of the structure, prescribe formal quality plans, covering both design and construction, that are supplementary to the control procedures prescribed in the other relevant Eurocodes.

In the absence of such formal quality plan the following apply:

Choice of materials and construction techniques shall be in compliance with the design hypothesis. The design documents shall indicate details, sizes and quality provisions as well as special devices and tolerances between structural and non structural elements.

Elements of special structural importance requiring special checking during construction shall be pointed out on the design drawings. In this case the checking methods to be used shall be also specified.

During construction the properties of the materials used shall be checked with respect to all the limits -lower and/or upper as prescribed.

3. Specific aseismic considerations regarding the ground

1. General
2. Ground Investigations
3. Special ground investigations

3.1 General

In regions which are divided into micro-zones the relevant rules shall apply. Elsewhere ground investigations as described in Clause 3.2 shall be carried out.

3.2 Ground Investigations

Site investigations shall be carried out in order to:

i) classify the soil in profile types according to Clause 4.2.1.2.

ii) identify possible ground instability that includes the possibility of liquefaction and slope instability.

3.3 Special ground investigations

Special ground investigations may be required for structures of high importance in high seismicity regions and for urban planning purposes. Guidance on the necessity of such investigations for the different types of structures are given in parts 1.3, 2 etc. of this Code.

As a minimum, sites shall be inspected by a specialist who shall investigate

i) evidences of recently active faulting

ii) the possible occurrence of large landslides affecting the area where structures are located and the presence of underground cavities.

Detailed description of the content, the procedures to be followed and the restrictions that may accompany these investigations shall be determined by the National Authorities.

4.1 Seismic zones

For the purpose of this Code, national territories shall be classified into seismic zones depending on the degree of local seismic activity.

The zonation of each country is made by the National Authorities and within each zone the peak ground acceleration in rock or firm soil a_g to be considered for design purposes is constant and shall also be defined by such Authorities.

4.2 Definition of the seismic action

For design purposes the seismic motion can be defined by means of various different models, whose complexity should be appropriate to the problem to be solved.

4.2.1 Response spectrum representation

Within the scope of this Code the surface earthquake motion is modelled by one or more normalized elastic response spectra for horizontal motion having a form appropriate for the zone and for rock or firm soil conditions. Normalization is made with respect to the peak ground acceleration a_g pertinent to the corresponding seismic zone and to rock or firm soil conditions.

The horizontal motion is described by two orthogonal components considered independent and represented by the same response spectrum.

Unless more reliable information is available, the vertical component of the earthquake vibration can be modelled by the same response spectrum as defined for horizontal motion scaled to 0.70.

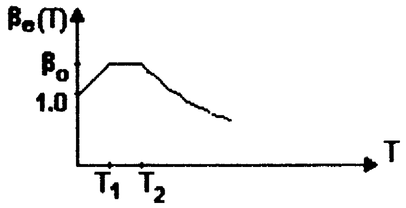
4.2.1.1 Normalised elastic response spectrum

The elastic response spectrum of acceleration $B_e(T)$ normalised to the peak ground acceleration a_g and corresponding to a viscous damping ratio of $\zeta = 5$ % is defined by the four parameters B_o, T_1, T_2, k and the following expressions:

$$0 < T < T_1 \rightarrow \beta_e(T) = 1 + \frac{T}{T_1} * (\beta_o - 1)$$

$$T_1 < T < T_2 \rightarrow \beta_e(T) = \beta_o$$

$$T > T_2 \_\_\_ \rightarrow \beta_e(T) = \left(\frac{T_2}{T}\right)^k * \beta_o$$



where:

T	is the vibration period of a linear single degree of freedom system.
Be(T)	is the spectral value of acceleration normalised to a_g
Bo	is the maximum value of the normalised spectral value assumed constant between T1 and T2.
T1, T2	are the limits of the constant spectral acceleration branch in the response spectrum.
k	is an exponent which influences the shape of the response spectrum for a vibration period greater than T2.

The values of these parameters are indicated in Table 1 in clause 4.2.1.2 for soil profile A.

The four parameter definition of the response spectrum does not model adequately the real vibration characteristics in the long period range (T > 5 sec). In cases where such period range of the action is important for the response of the structure results from ad hoc studies or properly motivated conservative assumptions shall be adopted.

4.2.1.2 Site dependent response spectra

The effects of local soil characteristics are accounted by the modification of the shape of the response spectrum. Three soil profiles are to be considered as follows:

Soil profile A

A1	Hard rock or soft rocky formations extended considerably both in area and in depth, provided that they do not show signs of intense cracking.
A2	Extended layers of particularly dense coarse-grained granular materials, with a small percentage of clay/silt contents.
A3	Extended layers of very stiff, strong, over-consolidated clay with high values of unconfined compressive strength.

Soil profile B:

- B1 Intensely in-situ eroded rock formations or soils that can be assimilated to them from a mechanical characteristics point of view.
- B2 Coarse-grained granular materials of medium relative density.
- B3 Medium stiff clay, slightly over-consolidated.

Soil profile C:

- C1 Loose coarse-grained granular materials with low relative density values.
- C2 Clay/silt soils of reduced stiffness.

Additions and/or modifications to the above classification may be given by the National Authorities so as to better conform to the different soil types prevailing in each Country.

The site dependent elastic response spectra of acceleration $\beta_s(T)$ normalised to the peak ground acceleration a (for rock in the same seismic zone) corresponding to a damping ratio of $\zeta = 5$ % is defined by the same parameters described in 4.2.1.1 plus the soil parameter S with the following expression:

$$\beta_s(T) = S * \beta_e(T)$$

The values of the parameters that define the spectra are specified by the National Authorities and are entered into Table 1.

Table 1

Soil Profile	S	T1(s)	T2(s)	k	βo	do (cm)
A	1	0.2	0.4	1	2.5	60
B	1	0.2	0.6	1	2.5	90
C	0.8	0.3	0.8	1	2.5	120

For sites with soil conditions not matching the three soil profiles described above, special studies for the definition of the seismic action are required.

4.2.1.3 Absolute ground displacements

The values of the absolute maximum ground displacement d are calculated using the following equation:

$$d = a * d_o$$

where d is the value included in Table 1 for each soil profile.

4.2.2 Power spectrum representation

The seismic motion may be described as a stationary process, defined by a power spectrum associated with a certain duration.

The power spectrum, may be expressed as the power spectral density function of the acceleration, shall be consistent with the elastic response spectrum defined in Clause 4.2.1.2.

4.2.3 Time-history representation

The seismic motion may be described by a set of artificially generated accelerograms. It has to be consistent with the elastic response spectrum defined in Clause 4.2.1.2 and the associated with a certain duration.

4.2.4 Spatial model of seismic motion

For structures whose general characteristics including one or more of the following overall dimensions, distance between contact points with the ground, foundation types, amount of embedment e.t.c. - do not allow to treat the seismic excitation as a point motion, a complete model of the seismic wave field shall be adopted.

This model shall include :

the surface motion, as defined in Clause. 4.2.1. The motion is assumed as isotropic in plan, and uncorrelated for the various directions. The vertical motion, also uncorrelated with the horizontal ones, has an intensity generally set (see Clause. 4.2.1) at |70 %| of the latter.

the direction, velocity of propagation and relative amplitudes of the body and surface waves, all compatible with the surface motion at a given point as described above.

The parameters of the wave field shall be defined on the basis of justifiable assumptions regarding the possible mechanisms of wave propagation from the far field to the site.

4.3a Behaviour Factor q

The capability of a structural system to resist seismic actions in the nonlinear range is accounted for by the possibility of designing it for forces smaller than those inherent to a linear elastic response.

For design purposes and in order to avoid the need for a nonlinear analysis, the concept of behaviour factor q is introduced. This parameter, which takes into account the energy dissipation capacity through ductile behaviour, is used to correct the results obtainable from a linear analysis in order to get an estimate of the nonlinear response. Accordingly, the determination of the internal forces for strength verifications may be based on the design spectra presented below.

The values of the behaviour factor q are given, for the various materials and structural systems and according to various ductility levels, in parts 1.3, 2 and following of this Code.

4.3 Linear analysis design spectra

The design spectra to be used in the analysis of structures are defined by the following expressions:

$$0 < T < T_1 \rightarrow \beta(T) = \alpha * S * \left(1 + \frac{T}{T_1} * \left(\frac{\eta * \beta_o}{q} - 1\right)\right)$$

$$T_1 < T < T_2 \rightarrow \beta(T) = \frac{\alpha * \eta * S * \beta_o}{q}$$

$$T > T_2\_\_\_ \rightarrow \beta(T) = \frac{\alpha * \eta * S * \beta_o}{q} * \left(\frac{T_2}{T}\right)^k$$

where : $\beta(T) \geq 0.20 * \alpha$

- α is the ratio of the peak ground acceleration to the acceleration of gravity
- η is a corrective factor defined in Clause 4.3.1 for structures with damping different from $\zeta = 5$ %.

q is the behaviour factor.

The values of parameters S, T1, T2and be may be taken from Table 1 presented in Clause 4.2.1.2 or be otherwise specified by the National Authorities and entered into Table 2.

Table 2

Soil Profile	S	T1(s)	T2(s)	k	βo
A					
B					
C					

4.3.1 Design spectra for other damping

Some structural systems may display a viscous damping different from the reference value adopted in clause 4.2.1.1.

In such cases the values of the design spectral ordinates shall be corrected by the factor:

$$\eta = \sqrt{\frac{5}{\zeta}} \ ; \ \eta > 0.70$$

where ζ is the value of the viscous damping ratio expressed in percent.

The values of ζ if other than 5 % are given, for the various structural materials, in parts 1.3, 2 and following of this Code.

5.1 Field of application.

The methods of analysis given in this chapter apply to all types of structural systems except for the restrictions explicitly indicated in parts 1.2, 1.3, 2 etc.

Simpler procedures may be used for the analysis of buildings and for other types of structures according to parts 1.2, 1.3, 2 etc. of this Code.

5.2 Modelling

The determination of the seismic effects on the structure shall be based on an idealised mathematical model which is adequate for representing the actual behaviour.

In general the model shall also account for a possible non-planar motion of a structure.

The model shall also account for all non-structural elements that can influence the response of the main resisting system.

If the structure can vibrate in two orthogonal. directions without significant coupling and without significant coupling between translational and torsional vibrations, it may by analysed by means of two separate planar models, one for each orthogonal direction.

5.3 Methods of analysis

The seismic effects and other action effects to be considered according to the combination rule given in Clause 6.2 of part 1.1 may be determined on the basis of a linear model of the structure as a whole.

The P-Δ effect shall be taken into account when necessary as specified in parts 1.2, 2 etc. of this Code. The stiffness shall also be determined as prescribed in those parts, and the masses in accordance with Clause 6.2. of part 1.1

The following types of analysis are considered in this Code:

a) Response spectrum analysis

b) Static analysis

c) Power spectrum analysis

d) Time domain dynamic analysis

5.3.1 Response spectrum analysis

The following types of responce spectrum analysis are considered in this Code:

i) Multi-modal response spectrum analysis

ii) Simplified dynamic analysis

5.3.1.1 Multi-modal response spectrum analysis

In a complete modal analysis the effects of seismic action shall be evaluated by using the design spectrum $\beta(T)$ according to section 4.3.

The responses of all modes of vibration making a significant contribution to the global response shall be taken into account. The modal responses for each variable relevant to the resistance or the stability of the structure shall be evaluated.

If the modal responses can be regarded as independent of each other, their combination will be performed according to the expression:

$$S = \sqrt{\sum S_i^2}$$

where S denotes the response quantity under consideration and S_i its value in the i-th mode of vibration.

For the purpose of application of this combination rule two modal responses may be considered as independent when the periods of the modes differ by more than 10 %.

If the modes cannot be considered to be independent more accurate procedures for the combination of the modal maxima shall be adopted.

For seismic effects of vectorial type (multi-component internal forces in a given section) the vector summation of each component over all the modes may be physically inconsistent and result in a large overestimation of the total effects. In those cases recourse is allowed to approximate conservative procedures.

5.3.1.2 Simplified dynamic analysis

This type of analysis can be applied to regular and compact structures, for which no essential contribution from higher modes of vibration to the action effects is expected.

In this analysis, the modal approach is restricted to the approximately evaluated fundamental mode of vibration or modes if more than one direction is relevant. The period of the first mode shall be calculated according to the ordinary methods of mechanics considering all elements contributing to the stiffness of the system. The seismic action effects shall be determined by applying horizontal forces F_i acting parallel to the seismic direction to all masses W_i.

The forces will be determined by assuming the entire mass of the structure as a substitute mass of the fundamental vibrational mode leading to the relation

$$F_i = \beta(T) * s_i * W_i * \frac{\sum_{j=1}^{N} W_j}{\sum_{j=1}^{N} s_j * W_j}$$

where

$\beta(T)$ is the ordinate of the design spectrum according to Clause 4.3 of part 1.1

W_i, W_j are the weights of the masses taken into account according to Clause 6.2 of part 1.1

$s_{i,j}$ are the corresponding displacements in a rational approximation for the fundamental mode shape of the structure.

5.3.2 Static analysis

For this analysis the effects induced by the seismic action shall be determined by applying to the structure a system of horizontal. forces which act parallel to the

assumed seismic direction and are applied at the centres of each mass they are given by the relation

$$F_i = \varepsilon_o * W_i$$

where

$\varepsilon_o = \frac{\alpha * \eta * S * \beta_o}{q}$ is a factor corresponding to the maximum value of the design spectrum according Clause 4.3. of part 1.1

W_i is the weight of the i-th mass taken into account according to Clause 6.2 of part 1.

5.3.3 Power spectrum analysis

A linear dynamic analysis of the structure shall be performed, either by using modal analysis or frequency dependent response matrices using as input the acceleration power density spectrum defined in Clause 4.2.2. of part 1.1

The resulting power density spectra of the response quantities of interest (internal forces and deformations) shall be used to obtain the design values of these quantities.

The elastic response values are evaluated first, defined as the same fractiles of the distribution of the response that are used in the definition of the elastic response spectrum (Clause 4.2.1 of part 1.1)

The design values are finally obtained by dividing the elastic response values above by the appropriate behaviour factor q.

5.3.4 Time domain dynamic analysis

For a direct dynamic analysis performed by numerical integration of the differential equations of motion time-histories for the ground motion, artificially generated according to Clause 4.2.3, of part 1.1should be used, in agreement with the National Authorities.

The number of required accelerograms shall be such as to give stable statistical representation (average and variance) of the response quantities of interest.

For non-linear type of analysis the models used for describing materials and elements behaviour shall be supported by adequate reference to experimental and/or theoretical knowledge.

It shall be demonstrated, that by using the time domain analysis at least the same overall reliability will be obtained as by the standard design procedure of this Code (Clause 5.3.1 of part 1.1).

5.4. Calculation of individual members (appendages) supported by the main structural system

For the calculation of individual members or appendages, such as window parapets, gables, antennas, mechanical appendages, bridge railings etc. detailed guidance is given in parts 1.3, 2 etc. of this Code.

If the main structural system is calculated by using the simplified dynamic analysis (Clause 5.3.1.2 of part 1.1), individual members or appendages shall be designed for those seismic loads which they would have to sustain if directly connected to the ground, multiplied by the factor

$$\frac{\eta * \beta_s(T)}{q}$$

where

ç is the damping corrective factor (Clause 4.3.1of part 1.1) of the main structural system

$\beta_s(T)$ is the ordinate of the normalised elastic response spectrum (Clause 4.2.1.2 of part 1.1 for the fundamental vibration period T of the main structural system

q is the behaviour factor (Clause 4.3 of part 1.1) of the main structural system.

For the calculation of the main structural system, the influence of the subsystem shall be modelled by the forces calculated according to formula (5.4)(see above).

5.5 Calculation of the displacements

According to the rules of this Code, the displacements induced by the design seismic action may be calculated taking account of the inelastic deformation of the structural system by means of the simplified relation

$$d_s = q * d_e$$

where

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

q is the behaviour factor (Clause 4.3of part 1.1)

d_e is the displacement of a point of the structural system determined by linear analyses based on the design spectrum.

6.1 Components of seismic action

1. Use of response spectra
2. Use of power spectra or of samples of time-histories derived thereof
3. Use of spatial model of seismic motion

6.1.1 Use of response spectra

Denoting by E the design seismic action defined by the response spectrum of Clause 4.2.1 of part 1.1 and by Ex, Ey the same action when applied along two arbitrarily chosen horizontal orthogonal axes x and y and by Ez the (reduced) action along the vertical axis z, the seismic input to be considered for the structural analysis shall consist of the following three combinations:

$$E_{x'} \pm \lambda * E_{y'} \pm \mu * E_{z'}$$

$$\lambda * E_{x'} \pm E_{y'} \pm \mu * E_{z'}$$

$$\lambda * E_{x'} \pm \lambda * E_{y'} \pm E_{z'}$$

with the sign of each component in the expressions above being the most unfavourable for the purpose at hand.

The values for λ and μ will be given by the competent National Authorities.

Simplified forms of the general combination above are possible, depending on structural configuration and material type, or both. The allowable simplifications are given in the relevant chapters of this Code.

6.1.2 Use of power spectra or of samples of time-histories derived thereof

If the seismic action is represented by means of the power spectra, or by the corresponding time-histories as defined in Clause 4.2.2 and 4.2.3 of part 1.1 the seismic input for the structural analysis shall consist of three simultaneously acting independent random processes (or samples thereof) acting along two arbitrarily chosen horizontal orthogonal axes x and y, and the vertical axis z, this latter process being proprietary scaled according to Clause 4.2.1. of part 1.1. Simplified formulations of the input as defined above are possible, depending on structural configuration and material type, or both. The allowable simplifications are given in the relevant chapters of this Code.

6.1.3 Use of spatial model of seismic motion

In lack of specific information, based on actual records and/or physical arguments, about the possible wave patterns travelling across the ground layers, simplified conservative spatial models of the seismic motion can be adopted.

In any case, the frequency content of the motion shall be in accordance with that of the elastic spectrum defined in Clause 4.2.1, of part 1.1 while suitable assumptions will be made on wave velocities, depending on average soil conditions at the site.

6.2 Combination of seismic action with other actions

When checking for the earthquake effects, the following combination with other actions shall be considered:

$$\pm \gamma_I * E + G + P + \sum_i \Psi_{2i} * Q_{ik}$$

where the symbols have the following meaning:

γ_I importance factor, defined in Clause 2.1, dependent on the importance category (Clause 6.3).

E design seismic action as defined in Clause 6.1.1 and Clause 6.1.2 of part 1.1 evaluated by taking into account the presence of all gravity loads appearing in the combination formula.

For the determination of E, variable loads Q_{ik} are multiplied by the factors Ψ2i. These factors take into account the probability of loads Qik not being present over the entire structure during the occurrence of the earthquake as well as the probability that during the occurrence of the earthquake they act at values smaller than their characteristic values.

Values of Ψ_{Ei} applicable for occupancy and environmental loads on buildings are given in Clause 3.6 of part 1.2.

G permanent loads evaluated at their characteristic values

P prestressing at its final value

Q_{ik} variable loads at their characteristic values

Ψ_{2i} combination factors affecting the variable loads, giving the values of Q_{ik} to be considered for design. Values of Ψ_{2i} applicable for occupancy and environmental loads on buildings are given in Clause 3.6 of part 1.2.

6.3 Importance categories

Due to the reliability differentiation (Clause 2.1) structures are classified into different importance categories.

To each importance category an importance factor γ_I is assigned.

Detailed guidance on the importance categories and the corresponding importance factors are given in Clause 3.5 of part 1.2 and in parts 2, 3 etc. of this Code.

Part 1.2

BUILDINGS IN SEISMIC REGIONS GENERAL RULES FOR DESIGN

1.1 Scope

This part of the Code is concerned with buildings. It contains some principles and application rules applicable in addition to those given in part 1.1 and shall be used in conjunction with the provisions of part 1. 3.

1.2 Assumptions units symbols and reference codes

The provisions given in part 1.1 apply.

2 Structural regularity

For the application of simplified methods of analysis and for the determination of the behaviour factors q, distinction should be drawn between regular and non regular buildings.

A building can be classified as regular when the conditions expressed in Clause 2.1 and Clause 2.2 are simultaneously satisfied.

2.1 Geometrical and structural layout in plan

The plan configuration is compact, i.e. it does not present divided shapes nor large recesses. When re-entrant corners or recesses exist their dimension does not exceed 25% of the building external dimension in the corresponding direction.

The structure of the building is distributed along an orthogonal mesh defining two main directions with similar stiffness.

The building has an approximately symmetrical plan configuration with respect to those two main orthogonal directions.

At any storey the distances (measured in the two main directions) between the centre of masses and the centre of stiffness do not exceed 15 % of the "resilience radius" defined as the square root of the ratio of the storey torsional and translational stiffness.

The in-plan stiffness of the floors is high enough, in comparison with that of the vertical structural elements, so that a rigid behaviour may be assumed. Furthermore, the floors should not present large holes hindering such assumption especially if they are located in the vicinity of the main vertical structural elements.

2.2 Vertical configuration

The stiffness and mass properties are approximately uniform along the building height.

In case of gradual setback along its height, the setback at any floor is not greater than 20 % of the previous plan dimension in the direction of the setback and symmetry about the vertical axis is preserved.

If a setback greater than 20 %, but not greater than 50 % and preserving symmetry, occurs within the lower 15 % of the total height of the building above the surrounding ground level (or above the level of application of the seismic excitation), it may still be classified as regular. In such case the structure of the base zone in the vertical projection of the upper storeys must be able to support at least 75 % of the shear forces that would develop in that zone in a similar building without the base enlargement.

In case of setbacks occurring only in one facade, the overall setback (sum of setbacks at all storeys) is not greater than 30 % of the plan dimension in the first storey and at any floor the individual setback is not greater than 10 % of the previous plan dimension.

3.1 Modelling

In general the structure may be considered to consist of a number of vertical resisting systems connected by horizontal diaphragms rigid in their plane. For buildings complying with the regularity requirements set out in chapter 2 the analysis can be made by using two planar models, one for each main direction.

Buildings not complying with such regularity requirements shall be analysed by means of three-dimensional models, which, if applicable and justified, may maintain the rigid floor assumption.

3.2 Methods of analysis

1 Regular buildings

Unless otherwise specified by the National Authorities -depending on the local seismicity and the importance of the building - or prescribed in part 1.3, for regular buildings having a fundamental period less than or equal to 2*T2 (Clause 4.2.1.2 of part 1.1) the simplified dynamic analysis (Clause 5.3.1.2 of part 1.1) with the additional specifications in Clause 3.3.1, , Clause 3.3.2, and Clause 3.3.3 can be used.

For regular buildings having a fundamental period larger than 2*T2 the multi--modal response spectrum analysis (Clause 5.3.1.1 of part 1.1) or equivalent procedures (Clause 5.3.3, 5.3.4 of part 1.1) shall. be used together with the additional specifications of Clause 1.3.3.

2 Non regular buildings

Buildings not complying with the regularity requirements set out in chapter 2, and not explicitly excluded from the scope of this Code, shall be analysed by means of the multi-modal response spectrum analysis (Clause 5.3.1.1 of part 1.1) or by equivalent procedures (Clause 5.3.3, and Clause 5.3.4 of part 1.1).

3.3 Use of the simplified dynamic analysis

The following issues must be resolved in order to carry out a simplified dynamic analysis

1) Distribution of the horizontal forces
2) Fundamental vibration period
3) Torsional effects

3.3.1 Distribution of the horizontal forces

The effects induced by the seismic action shall be determined by applying horizontal forces F at each floor, given by the expression

$$F_i = \beta(T) * z_i * W_i * \frac{\sum_{j=1}^{N} W_j}{\sum_{j=1}^{N} W_j * z_j}$$

where:

W_i is the weight of the i-th storey including permanent loads and variable loads multiplied by Ψ_{Ei} (Clause 3.6)

z_i is the height of the i-th mass from the level of application of the seismic excitation.

$\beta(T)$ is the ordinate of the design spectrum as defined in Clause 4.3 part 1.1 for the fundamental vibration period T according to Clause 3.3.2.

The horizontal loads determined in this way shall be distributed between the vertical elements according to their stiffness, assuming rigid floors-

3.3.2 Fundamental vibration period

For the purpose of determination the fundamental period to be used in conjunction with the simplified dynamic analysis approximate expressions based on structural dynamics (e.g. Rayleigh-formula) may be used

3.3.3 Torsional effects

The torsional effect induced by seismic actions shall be taken into account generally by using in calculations a three - dimensional structural model, adequate for the consideration of the coupling between translational and torsional vibrations.

When complete symmetry of stiffness and mass about one axis parallel to the direction of the seismic excitation exists, torsional effects may be accounted for by means of the following simplified procedure:

- The effects of the seismic action shall be evaluated by applying to a planar model of the structure the set of forces defined in Clause 3.3.1

- The resulting effects on each element shall then be amplified by a multiplication factor

$$z = 1 + 0.6 * \frac{x}{L}$$

where

x is the distance of the element under consideration from the floor centre of symmetry, measured perpendicularly to the direction of the seismic action considered

L is the total horizontal dimension of the building, perpendicular to the direction of the seismic action considered.

When this procedure is not applicable but the regularity requirements of chapter 2 are satisfied, the following approximate evaluation of torsional effects, still using a planar structural model for the determination of seismic forces, may be applied, provided the following conditions are satisfied:

1) The centres of stiffness of the individual storeys are approximately vertical above each other.

2) The centres of mass of the individual storeys which may not coincide with the centres of stiffness are also approximately vertical above each other.

If these conditions are satisfied, at each floor of the building the application point of the horizontal seismic force is assumed to be displaced from its nominal location in relation to the mass centre, perpendicularly to the direction of the considered

seismic action, by the most unfavourable of the two eccentricities:

$$\Delta e_{max} = e_1 + e_2 \_ or \_ \Delta e_{min} = -e_2$$

e_1	is the additional eccentricity taking account of the dynamic effect of simultaneous translational and torsional vibrations
e_2	is the accidental eccentricity of storey masses from their nominal location

Denoting by B and L the building dimensions parallel and perpendicular to the considered seismic action, the following approximations may be applied to e_1 and e_2

$$e_2 = 0.05 * L$$

$$e_1 = 0.1 * (L + B) * \sqrt{10 * \frac{e_o}{L}} \leq 0.1 * (L + B) \_ or \_$$

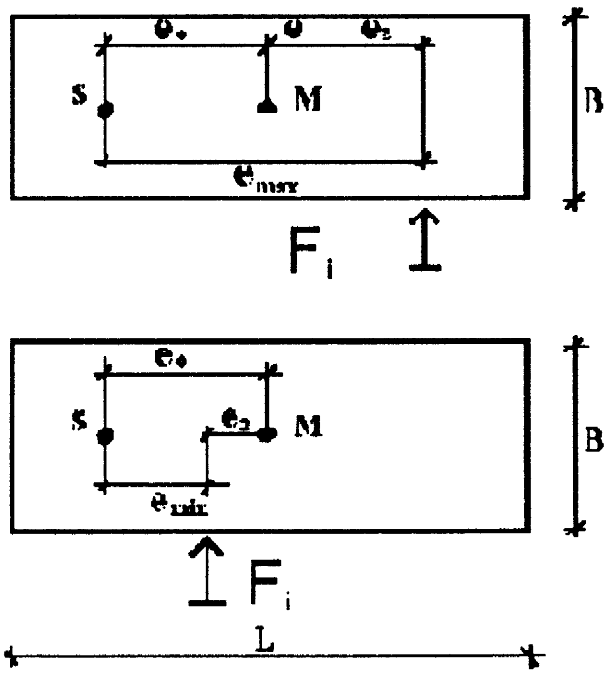
$$e_1 = \frac{1}{2 * e_o} * \left(l_s^2 - e_o^2 - r^2 + \sqrt{\left(l_s^2 + e_o^2 - r^2 \right) + 4 * e_o^2 * r^2} \right)$$

$$l_s^2 = \left(L^2 + B^2 \right) / 12$$

Where

e_o	is the actual geometric eccentricity (distance between the stiffness centre and a straight line through the mass centre running parallel to the seismic action considered)
L	is the total horizontal dimension of the building, perpendicular to the direction of the seismic action considered
B	is the total horizontal dimension of the building. parallel to the direction of the seismic action considered
r^2	is the ratio between torsional and translational stiffness of the structure.
e_1	is assigned the lower of the two values.

For $r^2 > 5 * (l_s^2 + e_o^2)$ the eccentricity e_1 may be neglected.



3.4 Components of the seismic action to be considered as acting simultaneously

In the case of buildings, the vertical component of the seismic action can be disregarded, except otherwise explicitly indicated.

Independently of other specific prescriptions, the vertical component shall be accounted for in the following cases, if α is greater than 0.2:

horizontal structural members spanning 20 meters or more

horizontal or inclined cantilever structures

having a significant arching behaviour

columns resting on beams.

For buildings which satisfy the regularity requirements set forth in Clause 2.1 and Clause 2.2 of part 1.2 the seismic action can be assumed to act separately along two main orthogonal horizontal axes of the structure.

3.5 Importance categories

Buildings are classified into 4 importance categories as defined by the National Authorities:

To each importance category correspond different values of importance factors γ_I, given by the National Authorities according to the following table:

Importance Factor

#	I	II	III	IV
γ_I	1.4	1.2	1.0	0.80

3.6 Combination factors for the variable loads

The combination factors Ψ_{2i} appearing in the combination formula of Clause 6.2 of part 1.1 shall be obtained from the following table, in which the specific values shall be taken from the Eurocode on Actions.

The combination factors Ψ_{Ei} introduced in Clause 6.2 of part 1.1 for calculating the inertia forces are given by the following expression

$$\Psi_{Ei} = \varphi * \Psi_{2i}$$

For multi-storey buildings the values of φ shall be obtained from the following table, in which the specific values shall be entered by the competent National Authority.

Type of variable load	Occupation of storeys		φ
Categories A-C	storeys independently occupied	top storey	1.0
		any other storey	0.5
Caregories A-C	some storeys having correlated occupancies	top storey	1.0
		storeys having correlated occupancies	0.8

	any other storey	0.5
Categories D-F archives	any storey	1.0

4.1 Safety against collapse (ultimate limit state)

The no-collapse requirement is considered satisfied if the following conditions are met:

i) Resistance capacity of the structural elements

ii) Ductility

iii) Overall stability

iv) Foundations

4.1.1 Resistance capacity of the structural element

Checking the resistance capacity of the structural element consist of checking:

1) Strength

2) Safety against 2nd order effects

4.1.1.1 Strength

The following relation must be satisfied for all structural elements

$$\frac{1}{\gamma_{rd}} *R_d - S_d *(\gamma_I *(E,G,P,\Psi_{2i},Q_{ik})) > 0$$

where:

Rd is the design resistance capacity of the component, calculated according to the rules specific to the material and according to the mechanical models which relate to the specific type of structural system as given in part 1.3 of this Code

γ_{Rd} is a partial coefficient relating to the type of element and the type of verification performed

Sd is the design action effect, induced by the combination of actions, 2nd order effects included, if necessary (Clause 4.1.1.2 of part 1.2).

4.1.1.2 Safety against 2nd order effects

The design interstorey drift dr shall be evaluated as the difference in the displacements at the top and bottom of the storey under consideration. (Calculated as per Clause 5.5 of part 1.1).

P-Δ (2nd order)-effects need not be considered when the stability coefficient È is limited according to the relation

$$Q = \frac{P_{tot} *d_r}{V_{tot} *h} £ 0.10$$

where:

Ptot is the total gravity load at and above the storey considered dr is the design interstorey drift

Vtot is the total seismic design shear action acting across the storey considered is the storey height.

4.1.2 Ductility

It must be verified that both the structural components and the structure as a whole possess sufficient ductility in accordance with the relevant provisions of part 1. 3.

4.1.3 Overall stability

The building structure regarded as a rigid body shall be stable under the set of forces given by the combination rules of Clause 6.2. of part 1.1. Herein are included the verifications of overturning, sliding and uplifting.

4.1.4 Foundations

The stability of foundations subjected to the action effects of the seismic combination shall be verified according to Eurocode No 7 and the rules specified in part 5 of this Code.

4.2 Limitation of damage (serviceability limit state)

The requirement for limiting damage is considered satisfied if contiguous buildings are separated by appropriate seismic joints and interstorey drifts are limited.

For the purpose of the required verifications the deformations of the building shall be determined according to Clause 5.5. of part 1.1

1. Seismic joints between structures
2. Limitation of interstorey drift

4.2.1 Seismic joints between structures

Unless otherwise specified in part 1.3, the following limitations apply:

1) The minimum distance between two contiguous structures shall not be less than

$$\min d_j = \frac{5}{100} + \frac{z}{200}$$

where

min dj is the minimum distance (in meters) at the height z in meters) above the level of application of the seismic excitation.

2) The distance between two contiguous buildings shall not, at any elevation, be less than the sum of the design deflections at the reference point concerned due to the design seismic action, divided by the factor v. This factor takes into account the different deformations due to the earthquakes with different probabilities of occurrence which are considered in the ultimate and the serviceability limit states. It shall be taken equal to 2.5

3) The above verification can be omitted if a joint having a width not less than max dj is provided,

where

$$max = \frac{5}{100} + \frac{z*q}{200}$$

4.2.2 Limitation of interstorey drift

The design interstorey drifts dr calculated as per Clause 4.1.1.2 of part 1.2 shall be limited, according to the types of non structural members used.

Unless otherwise specified in part 1.3 the following limits shall be observed:

- buildings having non-structural elements of brittle materials (e.g. tiles) attached to the structure.

$$d_r \leq v * 0.002 * h$$

- buildings having non-structural elements fixed in a way as not to interfere with structural deformations

$$d_r \leq v * 0.006 * h$$

where

h is specified in Clause 4.1.1.2 of part 1.2

v is specified in Clause 4.2.1. of part 1.2

Part 1.3

BUILDINGS IN SEISMIC REGIONS. SPECIFIC RULES FOR DIFFERENT MATERIALS AND ELEMENTS

1.1 Scope

Part 1.3 is concerned with specific rules for different. materials and elements used in buildings. It is applicable in conjunction with the provisions of parts 1.1, 1.2 and 5 of this Code and with the other Eurocodes.

The following chapters are covered:

2 Specific Rules for Concrete Structures

3 Specific Rules for Steel Structures

4 Specific Rules for Composite Structures

5 Specific Rules for Timber Structures

6 Specific Rules for Masonry Structures

7 Specific Rules for Mixed Structures

8 Specific Rules for Elements

1.2 Assumptions, units, symbols and reference codes

The provisions given in parts 1.1, 1.2 and 5 of this Code and the other Eurocodes apply. The provisions of this part serve only as additions or modifications to these rules.

SPECIFIC RULES FOR CONCRETE STRUCTURES

2.0 Notation

Within this chapter, the following (additional to Eurocode 2) symbols are used:

Ac	gross cross-sectional area of concrete
Ao	area of the core of a concrete section (sectional area of concrete after spalling of the cover)
Fres	residual response of a section or an element to action-effects (response after a certain number of deformation reversals)
Peff	Prestressing force contributing to the confinement of a beam column joint
Vdd	shear resistance of a R.C. section due to the dowel action of reinforcing bars
Vjh	horizontal acting design force on a beam-column joint V
Vjv	vertical acting design shear force on a beam-column joint
V_{M_R}	shear resistance of a R.C. wall due to flexural failure
VR	shear resistance of a R.C. wall due to shear failure
ΔM_{Sd}	additional acting bending moment of a column due to the frame/infill interaction
ΔV_{Sd}	additional acting shear force on a column due to the frame infill interaction
i_{os}	over strength ratio (=VR/VS where VR is the shear resistance of all vertical elements of a floor infills included, and VS is the acting design shear force)
α_r	regularity index (equal to the ratio of the minimum over strength ratio along the height of the building to the mean value of over strength ratios of all floors)
β_ϑ	coefficient reducing the elastic angular distortion of a building to account for the presence of infill walls
ε_{su}	uniform steel elongation at failure
ζ	ratio of the minimum to the maximum acting shear forces in a section (derived for seismic load combinations)
μ_f	coefficient of concrete-to-concrete friction
$\mu_{1/r}$	curvature ductility factor of a R.C. section
ν_d	normalised design axial force (NSd: Acfcd)
ω_{wd}	mechanical volumetric ratio of stirrups

Note

A large part of symbols used along this document are defined in the text; thus, they are not explained in this list too.

2.1 General

The following issues must be resolved:

1. Field of application
2. Criteria for the satisfaction of the fundamental requirements
3. Materials
4. Behaviour factors

2.1.1 Field of application

For the design of reinforced concrete structures reference is made to Eurocode 2. The following provisions complete or partially replace Eurocode 2

The present provisions apply to ordinary reinforced and prestressed concrete buildings, having structural resisting systems belonging to one of the three types as defined below:

Frame System: A system in which both vertical loads and lateral forces are resisted by space frames.

Wall System: A system in which both vertical loads and lateral forces are resisted by vertical structural walls, either single or coupled. A coupled wall is composed by two or more simple walls, connected in a regular pattern by adequately reinforced ductile beams ("coupling beams").

Dual System: A system in which support for the vertical loads is essentially provided by a space frame. Resistance to lateral action is contributed in part by the frame system in part by structural walls, single or coupled.

2.1.2. Criteria for the satisfaction of the fundamental requirements

The following Clauses develop the criteria set forth in Clause 1.2, of Part 1.1 so that R.C. structures comply with the corresponding requirements. Whenever needed, these criteria are supplemented by application rules cross-referenced to other chapters.

1. Non-collapse
2. Low damageability

2.1.2.1. Non-collapse

The fundamental requirement of Clause 2.1.1 Part 1.1 is deemed to be satisfied In the case of R.C. structures if the following criteria are applied simultaneously:

1) Local strength
2) Overall ductility (energy dissipation)
3) Specific additional measures
4) Quality assurance plans

2.1.2.1.1. Local strength

a) Under the design earthquake all critical regions should exhibit an action-effects' resistance adequately higher than the action-effects produced in these regions by the aforementioned earthquake.

b) Whenever more specific data are missing, it is allowed to be considered that under seismic conditions the design-value of "action-effects resistance" is equal to the design-strength value under monotonic loading, provided that all other provisions of this Code are met.

c) Second order effects shall also be taken into account.

2.1.2.1.2. Overall ductility (energy dissipation)

An adequate energy dissipation capacity of the structure, without a substantial decrease of its overall resistance against horizontal and vertical loading, should be secured. To this end, under the design earthquake. local ductility demands should be distributed to the largest possible areas of the entire structure, avoiding concentration in only few weak points. In this respect, three "Ductility Classes". "DC", are distinguished, leading to different behaviour factor q values; for each DC. separate application rules will be given for the detailed criteria presented here below :

a) Capacity design criterion

The possible inelastic dissipative mechanisms shall be favourably controlled. The criterion, besides the local ductility privations, is considered to be followed if the design of some appropriately selected critical regions is based not only on the action-effects bound by the analysis. but also on the strength required to satisfy equilibrium when adjacent members have reached their actual yield strengths. More specifically, in R.C. frames this criterion is applied as follows:

1) Plastic hinges should appear first in end cross-sections of beams than in columns.

2) The resistance of beam-column joints should be higher than the resistances of the end cross-sections of concurring beams and columns.

3) The design of critical regions should take into account the actual strength of the of the adjacent end cross-sections.

4) The ratio of the tensile strength to the yield strength of steel bars should also be appropriately limited; similarly, the actual steel strengths should not be disproportionately higher than the nominal ones (Clause. 2.1.3.2).

b) Local ductility criterion

The overall ductility of the structure necessitates high plastic rotational capacities of the critical regions of many structural elements. This criterion is deemed to be observed as follows:

(i) A sufficient curvature ductility is provided in all critical regions. When more precise date are missing, this rule is deemed to be satisfied if the conventional curvature ductility factor ("CCDF") of these regions (defined as the ratio of the curvature at the post-failure 85%-strength level over the curvature at yield of the tensional reinforcement), is higher than a specific value given in Clause. 2.3.1.3b. 2.3.2.3b, 2.3.3.3b and 2.6 for each of the Ductility Classes and structural elements.

For rectangular cross-sections of beams, columns and walls, it is allowed to use the following simplified expression of "CCDF":

$$\mu_{1/r} = \frac{\varepsilon_{cu}}{\varepsilon_{sy}} * \frac{1-\xi_{sy}}{\xi_{cu}}$$

$$\mu_{1/r} = \frac{\varepsilon_{su}}{\varepsilon_{sy}} * \frac{1-\xi_{sy}}{1-\xi_{su}}$$

where:

ε_{cu} concrete strain at post-peak 0.85 fck level in the $\sigma_c - \varepsilon_c$ diagram

ε_{sy} tensional steel strain at yield

ε_{su} uniform tensional steel strain at failure

ξ_{cu} normalised neutral axis depth at the post failure 85% strength level when concrete is critical

ξ_{sy} normalised neutral axis depth at tensional steel's yield

ξ_{su} normalised neutral axis depth when the tensile steel strain is critical

(ii) Flexural failure of frame building elements should precede their shear failure.

(iii) Local buckling of compressed steel within potential plastic hinge areas should be appropriately prevented.

(iv) Appropriate concrete and steel qualities shall be adopted in order to enhance the local ductility.

c) Structural redundancy

Higher degree of redundancy accompanied by a redistribution capacity should be sought.

d) Secondary resistances

Non-structural elements, not considered in the structural modelling, may also contribute to energy dissipation, provided that they are uniformly distributed throughout the structure, and appropriate measures were taken against possible local adverse effects due to the intersection between structural and non-structural organism.

2.1.2.1.3. Specific additional measures

In order to observe the non-collapse requirement within acceptable limits of reliability, appropriate measures should be taken to cover the uncertainties of the models used for analysis and behaviour determination, (Clause. 2.2.3 of Part 1.1).

a) Structural configuration

The overall configuration of the structure (i.e. in space distribution of masses, stiffnesses and strengths) shall be kept as uniform as possible.

In this respect, the following two "Regularity Classes" of R.C. buildings are distinguished for practical purposes, in addition to the provisions of pars 2.1. of Part 1.2.

On the basis of the characteristics of Tables I, II and III, a R.C. building may be categorised in Regularity Class Rh if its characteristics do not exceed more than two of the limits set forth by these tables for class Rh. A R.C. building may be cate-gorized in Regularity Class Rm if its characteristics do not exceed more than two of the limits set forth by these tables for class Rm.

R.C. buildings beyond these two regularity classes are not covered by this part of the Code.

[The Regularity Classes issue is still under discussion]

Notation

n number of storeys

T fundamental vibrational period of the building

$\delta_{fs.max}$ max. deflection of the floor system (diaphragm)

$\delta_{rs,mean}$ mean displacement of the R.C. skeleton at the floor system's level

$\delta_{i+1} - \delta_i$ inter storey drift

α_r regularity index

A_{fs} plan area of the floor system (diaphragm)

A_{op} area of the opening

$e_{tot,i}$ total eccentricity between mass and shear centres in direction (i)

e_s geometrical eccentricity

e_{add} additional eccentricity (*)

e_{acc} accidental eccentricity (*)

(*) for dynamic effects

b) Analysis

Appropriate methods of analysis (Clause. 3.2 of Part 1.2) and q-factors (Clause 2.1.4) should be selected in accordance with the degree of regularity of the structure.

c) Resistance uncertainties

(i) Certain minimum dimensions of R.C. structural elements should be respected, so that tolerated geometrical deviations (dimensions and placement of reinforcements) produce acceptable degree of uncertainties in predicting the structural behaviour of the element.

(ii) Due to the additional uncertainties related to the position of contra flexure point in R.C. beams under the design earthquake, a substantial percentage of the upper fibber reinforcements of beams at their end cross-sections should continue along the entire length of the beam.

(iii) In order to minimise the risk of lateral instability of R.C. elements. a limitation of minimum to maximum dimensions of linear elements should be observed.

(iv) Due to the disproportionately high uncertainties in calculating $P - \Delta$ effects under seismic effects, the horizontal deformability of R.C. columns should be appropriately limited.

(v) Possible non predicted moments' reversals should be balanced by minimum reinforcement provided at the relevant face of beams.

d) Ductility uncertainties

An appropriate minimum local ductility is needed in every part of R.C. structures subjected to seismic actions (independently of the q-values adopted in design), in order to counterbalance a part of models' uncertainty thanks to a higher redistribution capacity.

A minimum value of tensional reinforcement is needed in order to avoid brittle failures at the moment of cracking.

When the normalised axial force increases, local ductility is decreasing while its prediction becomes more and more uncertain; therefore, an appropriate limitation o! normalised axial force values is needed.

2.1.2.1.4. Quality assurance plans

In order to fulfil the non-collapse requirement, an appropriate quality assurance plan should be set-forth and followed, regarding the design documents, the materials and workmanship, as well as the use and maintenance of the structure, as foreseen in Clause. 2.2.3. of Part 1.1.

2.1.2.2. Low damageability

The fundamental requirement of Clause 2.1.2 of Part 1.1is deemed to be satisfied in R.C. structures if the following criteria are used simultaneously.

1 Local strength

Verificatio.ıs are required as in Clause.2.1.2.1.1.of Part 1.3

2. Limitation of deformations

a) In order to avoid the impact between adjacent R.C. structures (or parts of the structure) during the design earthquake, a seismic joint of adequate width should be provided.

This width may be calculated as in Clause. 4.2.1. Part 1.2.

b) Angular distortions of infill walls or claddings, as induced by the deformations of the structure under the design earthquake, should be kept lower than their critical distortion which corresponds to their maximum sheer capacity.

In R.C. buildings the inter storey drift used in the calculation of these angular distortions may be found in Clause. 2.4.of part 1.3.

2.1.3. Materials

1. Concrete

The use of concrete class lower than C16 for DC "L" or C20 for DC "M" and DC "H" is not allowed.

2. Steel for R. C.

Steel reinforcements should fulfil the additional rules of Table IV.

3. Prestressed steel

No limitations of mechanical characteristics of prestressed steel are needed for beams not rigidly connected to a R.C. framework.

2.1.4. Behaviour factors

The parameter q introduced in Clause. 4.3 of Part 1.1, to account for energy dissipation capacity and post-elastic resistance of the structure, takes the following maximum values, provided that R.C. structures are strictly observing the design criteria and rules included in this Part 3.2 of the Code.

(a) For T< T1 refer to Eq. 4.6. part 1.1

(b) When both coupled and uncoupled wells are included in the structural system, the selection of q-factor is made on the basis of the more resistant type of walls

(c) Higher values of q-factors, equal to those valid for coupled wall systems may be applied under the condition (Clause. 2.6.1.2 of part 1.3) that:

(d) Core structures are defined as those in which more than 65% of the total seismic force is resisted by R.C. core(s)

(e) R.C. structures are considered as behaving like inverted penduli when

1. the 50% of their mass is located in the upper 1/3 of the height of the building, end
2. the dissipation of energy takes place mainly in one isolated region.

(f) These values may be increased provided that the designer proves that a corresponding higher energy dissipation is ensured in the critical region,

2.2.0 Additional requirements

1. Beams
2. Columns
3. Masobry infilled RC Frames
4. Beam-Column joints
5. Walls

2.2 Beams

This chapter includes the aseismic design rules of R.C. beams depending on their ductility class:

1. Beams of ductility class "H"
2. Beams of ductility class "M"
3. Beams of ductility class "L"

2.2.1. Beams of ductility class "H"

For the design of beams of ductility class "H" the following issues need to be resolved:

1) Design action effects
2) Design strength evaluation and verification
3) Local ductility
4) Specific measures

2.2.1.1. Design action-effects

a) The acting bending moments shall be obtained from the analysis of the structure for the seismic load combinations (Part 1.1, Clause. 6.2); redistribution according to EC2Eurocode2_pop_topic is permitted.

b) The acting shear forces shall be determined in accordance with the capacity design criterion (Clause. 2.1.2.1.2): The equilibrium of the beam will be considered under the appropriate transverse load and a rationally adverse combination of the actual resistance moments of the end cross-sections.

These moments should account for the actual value of tensional steel cross-sections. as well as for the probability of steel stresses higher than their design values.

In order to take into account this over strength, an appropriate γ_{Rd}- factor should be used to increase the values of also end resistance moments, considering however the lower probability that all end cross-sections exhibit simultaneously the same over strength.

This γ_{Rd}-factor should counterbalance the partial safety factor of steel (γ_s- 1.15) and cover the probability of considerable hardening effects. In the absence of more precise data, it can be taken $\gamma_{Rd} = 1.25$

At each end cross-section, two values of acting shear force shall be calculated:

the maximum $V_{sd,max}$ and the minimum $V_{sd,min}$ corresponding to positive and negative resisting moments at hinges.

2.2.1.2. Design strength evaluation and verifications

1. The flexural strengths are evaluated as foreseen in, EC2.

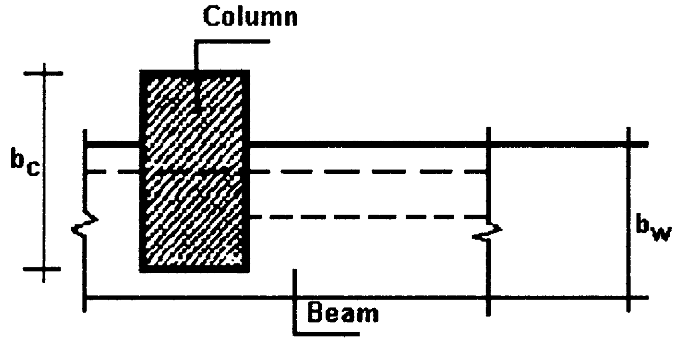
The flexural strength verification is carried-out as follows:

a) Observe the inequality

$$M_r > M_s(E,G,P,\sum \Psi_{2i}, Q_{ik})$$

b) The up-side reinforcements of the end cross-sections of T- or L- shaped beams shall be mainly placed within the beam's width; some of these reinforcements may be placed within parts of' slab, shorter than the flange width (which is drastically reduced because of local plasticity effects).

This slab width may be chosen as indicated here below



For exterior columns

Within the column width, in absence of transversal beam,

Within a length twice as long as the slab thickness on each side of beam, if there is a transversal beam of similar dimensions.

For Interior columns, these slab lengths may be respectively in-creased by $2d_f$

c) All reinforcements of the end cross-sections shall be appropriately anchored in order to resist bond degradations due to cyclic effects.

To this purpose, if more precise data are missing, the following deemed to satisfy rules should be observed:

(i) The favourable effect of column's compressive force for the bond of the horizontal bars crossing the joint is optimised if the width of the beam is not larger than the column's width increased on each side by half of the height of the beam.

However, $b_w \leq 2 * b_c$

(ii) In an event, 75% op the reinforcements of each side should be anchored inside the column or beyond the joint at a distance not smaller than $l_o = 1.5 * h_w$

(iii) The diameter of bars anchored beyond the joint should not be larger than (1/25)bc.

d) Extension of bars is allowed only outside the beam-column joints and outside the regions where plastic hinges may appear (Clause 2.2.1.3,of part 1.3) and under conditions insuring adequate bond resistance under cyclic actions.

To this effect, the following deemed to satisfy rules apply:

Along the necessary overlapping lengths (see EC2), hoops will be provided spaced at

$$s = \min\{h_w / 4, 100\text{mm}\}$$

Welded or sleeve-type joints should be staggered every 600 mm.

e) Efficient transfer of cyclic moments from beam to column shall be secured by reducing the eccentricity of beam relative to the columns into which it frames.

A deemed to satisfy rule is to limit the distance between the geometrical centrelines of the two members to less than bc /4

2. The shear strengths evaluation and verification are carried out as foreseen in EC2, with however the following modifications:

a) The term Vcc shall not be taken into account within the critical regions of potential plastic hinge formation (Clause. 2.2.1.3 of part 1.3).

b) The value V_{Rd2} in critical regions should not exceed 0,2 fcd.

Regarding the arrangement of shear reinforcements, two cases should be distinguished, depending on the algebraic ratio between the minimum end maximum design shear forces:

$$\zeta = V_{min} / V_{smax}$$

If -0.5 > æ > -1.0, i.e. if an almost full reversal of shear forces is expected, then:

(1) In case of a considerable probability of bidiagonal cracking caused by the negative V_{smin}, half of V_{smax} shall be retained by stirrups and half by bidiagonal reinforcements, while

(ii) In case of high probability of bidiagonal cracking caused by the negative, V_{smin} all V_{smax} shall be retained by bidiagonal reinforcements.

(*) In such a case, the verification is carried out by means of the Inequality

2.2.1.3. Local ductility

The criterion of Clause 2.1.2.1.2 is applied as follows

1. It shall be considered that a potential region for plastic hinge formation is a length twice as large as the beams depth next to the end cross-section (as well as a length twice the beam's depth on both sides of any other cross-section amenable to yield under the design earthquake conditions). Within these critical regions, if more precise data are missing, the required local ductility (Clause. 2.1.2.1.2) is satisfied if:

a) Hoops, of not less than 6 mm diameter, are provided with a spacing:

$$s_h = \min\{h_w / 4, 24 * d_h, 150\text{mm}\}$$

in order to secure an adequate confinement

$$s_h < 6 * d_l$$

so that local buckling of longitudinal bare is avoided.

The first hoop shall be placed at 50 mm from the end cross-section of the beam.

b) Additional reinforcements, not taken into account for the compliance to Equ. 2.2. are placed to the compressive zone, equal to half the amount of tensional reinforcement of these critical regions.

2. Besides, the necessary ductility conditions along the entire beam are deemed to be satisfied if:

a) The ratio of the tensional reinforcement is not

less than (Clause. 2.1.2.1.3)

$$\rho_{min} = \frac{1}{2} * \frac{f_{ctm}}{f_{yk}}$$

or greater than

$$\rho_{max} = \frac{1}{6} * \frac{f_{ck}}{f_{yk}}$$

b) There are at least two 14 mm in diameter bars at the top and two at the bottom side of the beam, running along its entire length.

2.2.1.4. Specific measures

The criterion of Clause 2.1.2.1.3, may be applied as follows;

(i) Minimum width of beams 200 mm

(ii) One fourth of the maximum top-side reinforcement should run along the entire beam's length.

(iii) Width to height ratio (cross-sectional dimensions) not lower than 0,25.

(iv) A certain minimum amount of reinforcement is needed in the lower side o P the end cross-sections to provide a reasonable flexural resistance against reversed action, even if no reversal is foreseen by the method of analysts used.

2.2.2. Beams of ductility class "M"

For the design of beams of ductility class "M" the following issues need to be resolved:

1) Design action effects
2) Design strength evaluation and verification
3) Local ductility
4) Specific measures

2.2.2.1. Design action-effects

The acting bending moments and shear forces shall be obtained from the analysis of the structure for the seismic load combinations (Part 1.1, Clause 6.2): redistribution according to EC2 is permitted.

2.2.2.2. Design strengths

1. The flexural strengths are evaluated as foreseen in EC2.

The flexural strength verification is carried-out as in Clause 2.2.1.2, with however, the following modifications:

Clause (2.2.1.2)1c(ii). lo = 100 mm

Clause (2.2.1.2)1c(iii): diameter < bc: 20

2 The shear strengths evaluation and verification is carried out as in Clause (2.2.1.2) 2 for DC "H".

2.2.2.3. Local ductility

The criterion of Clause 2,1,2.1.2 is applied as in Clause 2.2.1.3. with however the following modifications:

a) $s_h = \min\{h_w / 4, 24 * d_h, 200mm\}$

b) $s_h < 8 * d_l$

c) $\rho_{max} = \frac{1}{3} * \frac{f_{ck}}{f_{yk}}$

2.2.2.4. Specific measures

The criterion of Clause 2.1.2.1.3 is applied as in Clause. 2.2.1.4.

2.2.3. Beams of ductility class "L"

For the design of beams of ductility class "L" the following issues need to be resolved:

1) Design action effects
2) Design strength evaluation and verification
3) Local ductility
4) Specific measures

2.2.3.1. Design action-effects

The acting bending moments and shear force shall be obtained from the analysis of the structure for the seismic load combinations (Part 1.1, Clause. 6.2); redistribution according to EC2 is permitted

2.2.3.2. Design strength evaluation and verification:

1 The flexural strengths are evaluated as foreseen in EC2.

The flexural strength verification is carried-out as in Clause 2.1.21, with, however, the following modification:

Clause (2.1.2)1c(ii) does not apply

Clause (2.1.2)1c(iii): diameter < bc :15

2. The shear strengths evaluation and verification is carried out as in EC2.

2.2.3.3. Local ductility

The criterion of Clause (2.1.2.1)2b is deemed to be satisfied if the minimal conditions of EC2 concerning transversal reinforcement are covered and the following specific measures are respected.

2.2.3.4. Specific measures

The criterion of Clause 2.1.2.1.3 may be applied as follows:

a) Additional reinforcements, not taken into account for the compliance to Equ. 2.2, are placed in the compressive zone, equal to half the amount of tensional reinforcement of these critical regions.

b) The ratio of the tensional reinforcement is not

less than (see Clause 2.1.2.1.3c) $\rho_{min} = \frac{1}{2} * \frac{f_{ctm}}{f_{yk}}$

or greater than $\rho_{max} = \frac{1}{2} * \frac{f_{ck}}{f_{yk}} < 0.04$

2.3. Columns

This chapter includes the aseismic design rules of R.C. columns or linear elements not meeting the definition of beams.

In § 2.4. special design rules are presented, related to R.C. columns belonging to infill frames.

1) Columns of ductility class "H"
2) Columns of ductility class "M"
3) Columns of ductility class "L"

2.3.1 Columns of ductility class "H"

For the design of columns of ductility class "H" the following issues need to be resolved:

1) Design-action effects
2) Design-strengths-evaluation-and-verification
3) Local ductility
4) Specific measures

2.3.1.1 Design-action effects

1. The acting bending moments and axial forces shall first be determined from the analysis of the structure for the seismic load combinations. Second order effects should also be considered (Part 1.2, Clause 4.1.3).

a) However, in application of the capacity design criterion of Clause 2.1.2.1.2, in order to decrease the probability of plastic hinge formation in columns, the flexural action effects under seismic conditions shall be determined from the equilibrium of the column subjected to the most adverse combination of resisting moments of all adjacent beam end cross-sections.

b) Nevertheless, a relief of this rule is possible wherever the probability of full reversal of beam-end moments is relatively low.

c) The following or an equivalent practical rule has to be observed in applying the rule of Clause 2.3.1.1:

Taking into account the actual resistant moments of the end corrections of the beams framing into the joint, the equilibrium of this joint is considered. To this purpose, the following "sum of moments ratio" is found.

$$a_{CD} = \max \begin{cases} \dfrac{V_{RD}\,(|M_{Rd1}| + |M'_{Rd2}|)}{|M_{Sc1}| + |M'_{Sc2}|} \\[2ex] \dfrac{V_{RD}\,(|M'_{Rd1}| + |M_{Rd2}|)}{|M'_{Sc1}| + |M_{Sc2}|} \end{cases}$$

In this expression, M are the moments acting on columns as determined by the analysis, MRb are the actual resisting moments of the beam-ends taking into account the actual tensile steel area used (under the design stress fyd). The VRd factor, equal to 1,35 for DC "H" structures, retracts the stress-reduction Vs = 1,15, and accounts for the variability of fyk and for a probable degree of strain hardening. At ground level, the same values of "aCD " will apply at the lower part of columns (close to the foundations as at the upper part of these columns.

d) In applying the rule of Clause 2.3.1.1, the following method may be used :

The moments reversal factor $\delta = \dfrac{|M_{Sb1} - M_{Sb2}|}{|M_{Rb1}| + |M'_{Rb2}|}$ or $\dfrac{|M_{Sb1} - M_{Sb2}|}{|M'_{Rb1}| + |M_{Rb2}|}$

(in correspondence with the seismic direction valid for aCD) is used in order to estimate the magnification to be imposed on the bending moments MSc acting on the columns (as found by the analysis carried-out under the seismic load combination).

Thus, the capacity design criterion, is finally applied as follows:

$$(M_{Sc1})_{CD} = |1 + (\alpha_{CD} - 1)\delta|.M_{Sc1}$$

$$(M_{Sc2})_{CD} = |1 + (\alpha_{CD} - 1)\delta|.M_{Sc2}$$

where $1 + (\alpha_{CD} - 1)\delta > |q$.

An exemption from this criterion is permitted in the following cases:

In plane frames having four or more columns of the same structural importance, plastic hinging is allowed in one column for every three others remaining free from hinging.

In single and two-storey buildings, as well as in the top storey of multi-storey buildings columns hinge mechanisms are permitted.

The changing values of axial forces acting on a column when the seismic direction is alternating, shall be appropriately taken into account.

2. The acting shear forces shall be determined in accordance with the capacity design criterion Clause 2.1.2.1.2, considering the equilibrium of the column under the actual resisting moments of its end cross-sections.

$$(V_{Sc})_{C.D.} = V_{Rd} \frac{|M_{Rc}| + |M'_{Rc}|}{l_c}$$

where M denotes the actual resisting moments of the column-ends taking into account the actual tensile steel cross-sections (under the design stress fyd). The γRd, equal to 1,35 for DC "H" structures, retracts the stress-reduction γs = 1,15, and accounts for the fyk variability and for a probable degree of strain hardening.

2.3.1.2 Design-strengths-evaluation-and-verification

1. The flexural strengths are evaluated as foreseen in EC2 under the minimum value of acting axial force under seismic conditions.

The flexural strength verification is carried-out as follows:

Observe the inequality

$$M_R \geq (M_S)_{C.D.}$$

where (MS)C.D. denotes the acting moment found on the basis of

the capacity design criterion as described in Clause 2.3.1.1. In the verification, account should be taken of the bidimensional character of the seismic actions.

2. The shear strengths VR are evaluated as in EC2. The critical regions of potential plastic hinge formation are described in Clause. 2.3.1.3

Verification is carried-out by observing the inequality

$$V_R \geq (V_{SC})_{C.D.}$$

Regarding the arrangement of shear reinforcements, two cases should be distinguished, depend on the algebraic ratio between

the minimum and the maximum design shear forces

$$\zeta = \frac{V_{S\,min}}{V_{S\,max}}$$

If 0.5>ζ >-1.0, i.e. if an almost full reversal of shear

forces is expected, then:

(i) In case of a considerable probability of bidiagonal cracking caused by the negative V Smin, half of V Sax shall be retained , by stirrups and half by bidiagonal reinforcements, while

(ii) In case of a high probability of bidiagonal cracking caused by the negative Vsmin, all Vsmax shall be retained by bidiagonal reinforcements.

In such a case, the verification is carried-out by means

$$V_S \geq \sum A_s * f_{yd} * \sqrt{2}$$

2.3.1.3 Local ductility

The criterion of Clause 2.1.2.1.2 is applied as follows:

a) The length of the potential plastic hinges should be determined taking also into account the uncertainties related to this determination.

In the absence of more precise data, this length can be chosen as follows:

$$l_P = \min\{h_c, l_c / 5, 600\text{mm}\}$$

b) A minimum CCDF value of μ1/r = 15 should be secured, whereas the reduction of cross-section strength versus axial action effects because of the concrete "shell" loss under large compressive strains, should be counterbalanced by means of adequate confinement.

To these ends, as a deemed to satisfy rule the following inequality may be used:

$$\omega_{wd} \geq \max\{\lambda * (0.15 * \frac{A_c}{A_o} + \nu_d - 0.25), 0.20\}$$

where

ωwd = the mechanical volumetric ratio of confining hoops.

Ac = gross concrete area

Ao = core concrete area

vd = Ns*(Ac*fcd) normalised axial force

λ = 0,90 for circular hoops.

λ = 1.30 for "double hoop" rectangular pattern

λ = 1.10 for. triple hoop" rectangular pattern

c) The following minimal conditions should be respected:

(i) Hoops, of not less than 6 mm diameter, should be provided with a spacing bo/4 in order to secure a minimum ductility,

Sh = min {bo/4,100 mm} independently of the specific design parameters

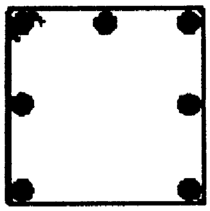
Sh < 6dL and dbh > 0.40*dbL*(fydL/fydh)^0.5 ,so that local buckling of longitudinal bars is avoided

(ii) Hoops' patterns should be selected in such a way that the largest part of the cross-section of the column would benefit of the triaxial effect produced by hoops and cross-ties.-

Since transversal forces are acting to concrete only at points where hoops are forming an angle (practically not larger that 135), or where appropriate cross-ties are anchored on longitudinal bar these "restraining" points should not be provided at distances larger than 200 mm.

Besides this restraining contributes to the avoidance of local buckling of longitudinal bars

For DC "H" structures the following confinement pattern should not be used in columns.



(iii) Acting axial forces should be limited by the rule

$$\nu_d = (N_d)_{max} / A_c / f_{cd} \leq 0.35 \quad (2.14)$$

since for higher values the implementation of the required CCDF becomes unpractical.

d) At least one intermediate bar (between corner bars) should be provided at each column side.

e) The total longitudinal reinforcements' ratio should not be less than 0,01 (in order to secure appropriate substitution of cracked concrete under unexpected tensional effects.), and not higher than 0.04 (in order to enhance rotational capacity at critical regions).

2.3.1.4. Specific measures

The criterion of Clause. 2.1.2.1.3 may be applied as follows:

(i) Minimum width of columns 300mm.

(ii) Unless special studies of P-Δ effects are carried-out, the width of the column should not be smaller than one eighth of the greater distance between the point of contraflexure and the ends of the columns (frame analysis parallel to the column width considered).

(iii) In the lower two storeys of buildings, half of the minimal transverse reinforcement of Clause. 2.3.1.3should also be provided after the critical regions for an additional length equal to half the length of these regions.

(iv) Infill masonry walls' to R.C. frame interaction effects should be appropriately taken into account.

(v) In the lower part of a ground floor column if above the footing there are resistant earth fills, or pavements able to restrain the deformability of the column, the critical region reinforcement should be appropriately extended.

2.3.2. Columns of ductility class "M"

For the design of columns of ductility class "M" the following issues need to be resolved:

1) Design action effects
2) Design strength evaluation and verification
3) Local ductility
4) Specific measures

2.3.2.1 Design action-effects

1. The acting bending moments and axial forces shall be determined as in Clause 2.3.1.1. with however a value $V_{Rd} = 1.20$

Local authorities may foresee additional conditions under which the application of this criterion might be substituted by directly given magnification factors.

2. The acting shear forces shall be determined as in Clause 2.3.1.1 with however a value $V_{Rd} = 1.20$

2.3.2.2 Design-strengths evaluation and verifications

Strengths' evaluation and safety verifications are carried-out as in Clause. 2.3.1.2.

However, in flexural moments' verification, bidimensional bending may be considered in an appropriately simplified way.

2.3.2.3 Local ductility

The criterion of Clause 2.1.2.1.2 is applied as follows:

a) The lengths of the critical regions of potential plastic hinge formation to be considered close to the end cross-sections of the column, may be taken as follows:

lp = min(hc,lc/6, 450mm)

b) A minimum CCDF value of ul/ = 6 should be secured, whereas the reduction of cross-sectional strength versus axial action effects because of the concrete "shell" loss under large compressive strains, should be counterbalanced by means of adequate confinement.

To these ends, as a deemed to satisfy rule, the inequality (2.13) may be used, taking however

for circular hoops $\lambda = 0.70$

for a "single hoop" rectangular pattern $\lambda = 1.65$

for "multiple hoops" rectangular patterns $\lambda = 1.00$

c) The following minimal conditions should be respected:

(i) Hoops, of not less than 6 mm diameter, should be provided with a spacing

$$s_h = \min\{b_o / 3, 150\text{mm}\}$$

in order to secure a minimum ductility, independently of the specific design parameters.

$$s_h < 8 * d_L \text{ and } \quad d_{bh} > 0.35 * d_{bL} * \sqrt{\frac{f_{ydL}}{f_{ydh}}}$$

so that local buckling of longitudinal bars is avoided

(ii) Since transversal forces are acting to concrete only at points where hoops are forming an angle (practically not larger than 135°) or where appropriate cross-ties are anchored on a longitudinal bar, these "restraining" points should not be provided at distances larger than 200 mm.

(iii) Acting axial forces should be limited by the rule

$\nu_d = (N_d)_{max} : A_c * f_{cd} < 0.4$ since for higher values the implementation of the required CCDF becomes unpractical.

d) At least one intermediate bar (between corner bars) should be provided at each column side.

e) The longitudinal reinforcements' ratio should not be less than 0.01, in order to secure appropriate substitution of cracked concrete under unexpected tensional effects, and not higher than 0.04 in order to enhance rotational capacity at critical regions.

2.3.2.4 Specific measures

The criterion of Clause 2.1.2.1.3 may be applied as follows:

(I) Minimum width of columns 250 mm

(ii) Unless special studies of P-b effects are carried-out, width of the column should not be smaller than one tenth of the greater distance between the point of contra flexure and the ends of the column (frame analysis parallel to the column width considered).

(iii) The clause Clause 2.3.1.4 is applied only in the lower first storey of buildings.

(iv) Infill masonry walls' to R.C. frame interaction effects should be appropriately taken into account.

(v) The rule of Clause 2.3.1.4 is also valid.

2.3.3 Columns of ductility class "L"

For the design of columns of ductility class "L" the following issues need to be resolved:

1) Design action effects
2) Design strength evaluation and verification
3) Local ductility
4) Specific measures

2.3.3.1. Design action-effects

The acting bending moments and axial forces as well as the shear forces shall be determined from the analysis of the structure for the seismic load combinations.

2.3.3.2 Design strengths evaluation and verification

1- Strengths' evaluation and safety verifications are carried out as in EC2 under the minimum value of acting axial force under seismic conditions.

In the verification, account should be taken of the bidimensional character of the seismic actions in a appropriately simplified way (e.g. as in Clause 2.3.2.2)

2.3.3.3 Local ductility

The criterion of Clause 2.1.2.1.2 is applied as follows:

a) The length of the potential plastic hinges should be determined taking also into account the uncertainties related to this determination.

In the absence of more precise data, this length can be chosen as follows:

$$l_p = \min\{h_c, l_c / 6, 450mm\}$$

b) A minimum CCDF value of μ1/r = 3 should be secured, whereas the reduction of cross-sectional strength versus axial action effects because of the concrete "shell" loss under large compressive strains, should be counter balanced by means of adequate confinement:

To these ends as e deemed to satisfy rule, the following inequality may be used

$$\omega_{wd} > 1,45 * \left(0.10 * \frac{A_c}{A_o} + \nu_d - 0.37\right)\_for\_single\_hoop\_pattern$$

$$\omega_{wd} > 0.10$$

$$\omega_{wd} > 0.75 * \left(0.15 * \frac{A_c}{A_o} + \nu_d - 0.45\right)\_for\_multiple\_hoop\_pattern$$

$$0.10$$

c) The following minimal conditions should be respected:

(i) Hoops, of not less than 6 mm diameter, should be provided with a spacing

(ii) Since transversal forces are acting to concrete only at points where hoops are forming an angle (practically not larger than 135°), or where appropriate cross-ties are anchored on a longitudinal bar, these "restraining" points should not be provided at distances larger than 200 mm.

(iii) Acting axial forces should be limited by the rule

$$\nu_d = (N_d)_{max} : A_c * f_{cd} < 0.45\_\_(2.18)$$

since for higher values the implementation of the required CCDF becomes unpractical

d) At least one intermediate bar (between corner bars)should be provided at each column side.

e) The total longitudinal reinforcements ratio should not be less than 0.01 (in order to secure appropriate substitution of cracked concrete under tensional effects), and not higher than 0.04 (in order to enhance rotational capacity at critical regions).

2.3.3.4 Specific measures

The criterion of Clause 2.1.2.1.3 may be applied as follows:

(i) Minimum width of columns 200 mm

(ii) Unless special studies of P-Δ effects are carried-out the width of the column should not be smaller than one tenth of the greater distance between the point of contra flexure and the ends of the column (frame analysis parallel to the column width considered).

2.4. Additional design measures for masonry infilled R.C. frames

This chapter contains the modified design measures needed in the case of infilled R.C. frames.

1) Requirements and criteria
2) In-plan irregularity due to infills
3) Consequences of the vertical irregularity due to infills
4) Design seismic forces
5) Analysis
6) Local effects of infill walls
7) Safety of the infill walls per se
8) Low damage ability requirements

2.4.1 Requirements and criteria

1 The consequences of possible in-plan-irregularities produced by the infills should be considered. To this end, a practical rule is given in Clause 2.4.2 (increase of eccentricity for torsional effects).

2 The consequences of the possible in-elevation-irregularity produced by the infills should be considered; vertical discontinuities of overstrength (possible soft stories) may result in disproportionate local ductility demands. In order to account for such an event decrease of the q-factor is imposed Clause. 2.4.3

3 The modified response of a R.C. structure because of the stiffening effect of the infills shall be considered, taking however into account the alleatoric behaviour of the infills (namely the variability of their mechanical properties, the possible modifications of their integrity during the use of the building, as well as the non-uniform degree of their damage during the earthquake itself). To this end, Clause 2.4.4. and Clause 2.4.5. aim deemed to satisfy design rules.

4 The uncertainties of the structural model of an infilled R.C. building should be adequately covered. In Clause. 2.4.5 relevant design rules are given imposing one or two analytical models depending on the in-elevation-regularity of the building.

5 The possibly adverse local effects because of the frame/infill interaction should be taken into account. Relevant design rules are given in Clause. 2.4.6.

6 The damage ability of the infills should be appropriately considered both regarding the risk to persons and the possibility of repair after the design earthquake. Relevant rules are given in Clause. 2.4.7 and Clause 2.4.8.

2.4.2 In-plan irregularity due to infills

a) In case of low in-plan-irregularity due to non-uniform distribution of infill walls in-plan, an additional eccentricity $\Delta e = 0.05l$ will be taken into account for calculation of the additional torsional effects due to the infills ("l" denotes the length of the floor in the direction under consideration).

b) In case of high in-plan-irregularity, due to non-uniform distribution of infill walls in-plan, torsional effects due to infills should be taken into account by means of an appropriate calculation

c) In this context, the following degrees of in-plan-irregularity have been considered:

-Low irregularity: When infills are arranged in a practically symmetrical way in-plan.

-High irregularity: The infills are arranged in a substantially unsymmetrical way in-plan, mainly along two consecutive faces of the building.

2.4.3 Consequences of the vertical irregularity due to infills

2.4.3.1 The vertical irregularity of a building taking also into account the actual infills, may be quantified as follows:

(a) An "over strength index", ios, is calculated for each separate floor of the building:

$$(i_{os})_k = \frac{V_R}{V_S} \quad (2.19)$$

where

Vs is the acting shear force for the considered floor, k, under the seismic load combination

VR is the available shear resistance of all vertical elements of the floor under consideration, which may be calculated as

$$V_R = \sum_m V_{Rc} + \sum_n V_{Rw} = 10 * \sum_m A_c * \tau_{Rd} + \frac{1}{V_w} * \sum_n A_w * f_{vko} \quad (2.20)$$

VRc and VRw denote the shear resistance of columns and infill walls, respectively.

Ac and Aw denote the horizontal cross sectional area of columns and infill walls, respectively.

$$\tau_{Rd} = \frac{1}{4} * f_{ctk0.05} * \gamma^{-1}$$

fvko is the characteristic value of the shear strength of masonry

γm is the partial safety factor for masonry.

The reduced shear resistance of infill walls with openings shall be appropriately taken into account.

(b) The main value of over strength indexes (ios) for all floors is calculated

(c) A "regularity index" ár of the building is calculated:

$$\alpha_r = (\min i_{os}) : i_{os} > 0.55$$

2.4.3.2 The values of behaviour factors, q, given in Clause 2.1.4 should be modified as follows, to take into account the increased ductility demand because of the over strength discontinuities of the building:

$$q' = (1 - \alpha_r) + \alpha_r * q \quad (2.22)$$

However, if ár > 0.85, the q-values in Clause 2.1.4 may be directly used.

2.4.4 Design seismic forces

The design shear forces shall be modified because of the decrease of natural period of reinforced concrete structures after the addition of infills.

If a more precise analysis is not available, the relevant rules included in Clause 2.4.5 may be used.

The calculation of the natural period of infilled structures may not be needed if the natural period of the bare one is close to the value T2 of the design spectrum (Part 1.1, Clause 4.2.1.1), if this is the case the maximum ordinates of the design spectrum will be considered anyway.

If the natural period of the bare structure is considerably higher than T2, the calculation of the natural period of the infilled structure may be made by means of a simplified method, e.g. a closed formula.

2.4.5 Analysis

(a) When the regularity index, a, is larger than 0.75, only the bare structure 's analysed, taking however into account design seismic forces corresponding to a natural period equal to the mean value of the natural period of the bare structure and of the natural period of the infilled structure.

Nevertheless, the possibly adverse local effects of infill walls on the columns of the structure should be avoided (Clause 2.4.6.)

(b) When the regularity index, αr , is smaller than 0.75, the analyses should be carried out as follows:

I) Mixed structure, with infill walls taken into account in their uncracked condition, and for design seismic forces corresponding to the decreased natural period of the mixed structure.

To this end, an appropriate model shall be used, taking also into account :

The uncertainties of wedging of the infill along the panel. From this point of view, diagonal strut models are deemed as more realistic than continuum models considering full perimetric bond.

Masonry panels containing openings, may be modelled by means of an appropriate increase of stiffness of the adjacent R.C. columns and beams (equivalent frame method).

The variability of the mechanical properties of the masonry. To this end, assumptions unfavourable for the R.C. elements should be made, taking into account characteristic values of elastic moduli, etc.

ii) Without infills (bare frame) and for design seismic forces corresponding to a natural period equal to the mean value of the natural period for bare structure and of the natural period of the infilled structure.

The R.C. frames elements are designed for the most unfavourable combination of action-effects resulting from analyses (i) and (ii).

2.4.6. Local effects of infill walls

The premature shear failure of columns under possible additional shear forces due to the diagonal strut action, shall be avoided. To this purpose, columns should be appropriately reinforced against shear in the regions where these additional shear forces are expected to act.

(a) Normally, in case of regular or slightly irregular structures ($1 > \alpha r > 0.85$) with spans fully filled with masonry, the transverse is reinforcement (closed stirrups) provided in critical regions of columns is sufficient.

(b) In case of irregular structures ($0.85 > \alpha r > 0.55$) the evaluation of the additional shear forces should be m de by means of an appropriate analysis.

(c) In case of spans filled with masonry walls the height of which is smaller than the floor height, the columns are considered as critical regions along the whole height and they are reinforced with the amount of stirrups required for the critical regions.

In case of DC "H" structures, the possible reductions of the final shear ratio of columns, taking into account their interaction with infill walls, should be considered.

In case of actual shear-ratio values lower than 4, columns should be reinforced as in Clause 2.4.5

2.4.7 Safety of the infill walls per se

No special design of infill walls is required.

However, in order to avoid brittle failure and premature disintegration of infill walls, as well as their out-of-plane falling of masonry blocks, light wire meshes, inadequately anchored into the walls, should be provided on both faces of infills.

2.4.8 Low damageability requirements

a) To cope with the general requirement of Clause 2.2.2 of Part 1.1, the inter storey drift of masonry infilled R.C. buildings should be appropriately limited, so that under the design earthquake conditions masonry cracking be not extended beyond a limit of easy repair.

The maximum expected inter storey drift Θs under the design earthquake, if a more precise method is not used, may be calculated as follows :

$$\Theta_S = (\beta_\Theta * \Theta_{el}) * q \text{\_\_\_\_} (2.23)$$

where,

β_θ denotes the reduction of inter storey drift of the bare structure, because of the residual stiffening effect of the infill walls; this factor is equal to unity for bare structures or for infilled dual systems, and may be taken equal to 2/3 in case of infilled frames.

Θel the inter storey drift of the bare structure calculated elastically

q the behaviour factor used in the determination of the seismic forces

b) The verification is made by taking as critical angular deformation of masonry the limit Θu which corresponds to the ultimate condition (maximum shear response) of confined masonry.

2.5. Beam-Column joints

The area included between the edges of a beam and a column framing to each-other should be appropriately designed following the criteria presented in Clause. 2.1.2.1.

1. Beam-column joints in DC "H" strutures
2. Beam-column joints in DC "M" structures
3. Beam-column joints in DC "L" structures

2.5.1. Beam-column joints in DC "H" strutures

For the design of beam-column joints in strutures of ductility class "L" the following issues need to be resolved:

1. Design action-effects
2. Design strength evaluation and verification

3. Local ductility

4. Reduced models' uncertainty

2.5.1.1. Design action-effects

The shear forces acting around the core of the joint shall be determined taking into account the most adverse conditions under seismic loading, i.e. capacity design conditions for the concurring beam-ends and the lowest compatible values of axial and shear forces of the framing elements.

Simplified expressions of these shear forces may be used under well defined conditions.

In every case, the absolutely maximum shear forces acting on joints shall be determined, corresponding to the appropriate seismic direction along the frame considered.

2.5.1.2. Design strength evaluation and verification

1. The shear-force transfer across the core of the joint can be effectuated through the following two mechanisms depending on whether the flexural cracks of the adjacent beam-ends can or can not be closed Curing a subsequent reversed bending moment:

   a) Diagonal strut:

   When at end-beam cross-sections, small width flexural cracks (due to a previous small amplitude moment-reversal) are subsequently closed, horizontal compressive forces are transferred through the concrete compressive zone and are combined with the vertical forces of the compressed zone of the column. Thus, a diagonal compressive strut is formed self-equilibrated within the joint.

   In such a case, the compressive strength of concrete (under simultaneous transversal tension) is governing the bearing capacity of the joint. In the absence of more precise data. the following inequality of Equ. (2.24.) should be observed:

   b) Truss and struts:

   When at end-beam cross sections, large width flexural cracks [due to a previous large amplitude moment-reversal) cannot be subsequently closed, the horizontal compressive forces may be transferred only through the reinforcement of the beam. Thus, a complete diagonal strut cannot develop; besides, yield penetration at both sides of the bar results in high and concentrated bond stresses alone its middle. Hence, extensive diagonal cracks within the core of the joint cannot be excluded; an

   additional truss mechanism is then needed for shear transfer.

   In such cases, an appropriate combined model should be used, in which diagonal concrete cracking shall be adequately limited. Simplified design methods covered by

   recognised experimental data are sufficient to this purpose.

2. The strength verification of beam-column joints of DC "H" structures shall be carried-out as follows:

   a) The Integrity of the diagonal strut defined in Clause. (2.5.1.2)1a shall be secured by means of the following inequality.

   $$V_{jh} < 20 * \tau_{Rd} * b_j * h_c \_\_\_(2.24)$$

   where the effective joint width bj, shall be taken equal to

   $$b_j = \min\{b_c, b_w + 0.5 * h_c\}\_\_\text{when}\_\_b_c > b_w$$

   $$bj = \min\{b_w, b_c + 0.5 * h_c\}\_\_\text{when}\_\_b_w > b_c$$

   In the case of an eccentricity "e" between the centrelines of beam and column.

$$b_j = \frac{1}{2} * (b_w + b_c + \frac{1}{2} * h_c) - e$$

b) An appropriate amount and shape of joint reinforcements shall be provided, in order to secure a safe shear transfer under the considered seismic conditions. Limited concrete cracking is allowed; to this purpose, the maximum tensile stress of concrete should not exceed the design value of the mean tensile strength of concrete.

$$\max \sigma_{ct} < \frac{f_{ct,m}}{\gamma_c}$$

In the particular case when Peff=0 and adequate intermediate vertical reinforcement is available in the column (so that at least Asvi/hj=(2/3)*Ash/hjw) the required horizontal reinforcement of the joint may be approximately calculated by means of the following expression:

$$\frac{A_{sh} * f_{yd}}{b_j * h_{jw}} \geq \frac{(2/3) * \gamma_{Rd} * (A_{s1} + A_{s2}) * f_{yd} - V}{b_j * h_{jc}} - \sqrt{\tau_{Rd} * (12 * \tau_{Rd} + v_d * f_{cd})} \quad (2.29)$$

(where $v_d = N_c / A_c / f_{cd}$) and

$$A_{sv,i} \geq \frac{2}{3} * A_{sh} * h_{jc} / h_{jw}$$

Besides, the concrete shear resisting term should be adequately reduced to account for the cyclic effects.

2.5.1.3. Local ductility

Beam-column joints are not considered as substantial energy dissipation areas.

However, the design criteria of Clause. 2.5.1.2 are deemed to secure a sufficiently constant response of joints under the design earthquake conditions.

2.5.1.4. Reduced models' uncertainty

The criteria of Clause 2.1.2.1.3 may be applied by providing minimum joint reinforcements as in the critical regions of columns.

Thus, the horizontal confinement reinforcement in beam column joints shall be diameter of 6 mm every sh=min(hc/4, 100 mm); If framing beams are present from all four faces of a column, the spacing of these hoops may be reduced to hc/2 but in no case larger than 150 mm. Besides, at least one intermediate (between column-corners) vertical bar shall be provided at each side of the joint; the maximum distance between consecutive bars should however be equal to 150 mm.

2.5.2. Beam-column joints in DC "M" structures

For the design of beam-column joints in strutures of ductility class "M" the following issues need to be resolved:

1. Design action-effects
2. Design strength evaluation and verification
3. Local ductility
4. Reduced models' uncertainty

2.5.2.1. Design action effects

As in Clause 2.5.1.1, taking however $\gamma rd = 1.15$

2.5.2.2. Design strength evaluation and verification

1. The shear transfer mechanism across the core is the same as in Clause 2.5.1.2
2. The strength verification of beam-column joints of DC "M" structures shall be carried out as follows:
   a) The integrity of the diagonal strut shall be secured as in Clause 2.5.1.2.
   b) Joint reinforcements shall be provided as described in Clause 2.5.1.2 for DC "H" structures. However, due to the lower number of large amplitude cyclic reversals expected to act In DC "M" structures (compared to those of DC "H" structures), the reduction factor of concrete shear resistance may be taken equal to 0.80.

   Thus, under the conditions of Clause 2.5.1.2, Equ.(2.29) is modified as follows:

$$\frac{A_{sh} * f_{yd}}{b_j * h_{jw}} \geq \frac{(2/3) * \gamma_{Rd} * (A_{s1} + A_{s2}) * f_{yd} - V}{b_j * h_{jc}} - 1.2 * \sqrt{\tau_{Rd} * (12 * \tau_{Rd} + v_d * f_{cd})} \quad (2.29)$$

   where $\gamma_{Rd} = 1.15$

2.5.2.3. Local ductility

As per Clause 2.5.1.3

2.5.2.4. Reduced models uncertainty

The criteria of Clause 2.1.2.1.3 may be applied by providing minimum joint reinforcements as in the critical regions of columns.

Thus, the horizontal confinement reinforcement in beam-column joints shall be a diameter of 6 mm every sh = min (hc/2, 150 mm). Besides. at least one intermediate (between column-corners) vertical bar shall be provided at each side of the joint.

2.5.3.-Beam-column joints in DC "L" structures

The horizontal confinement reinforcement in beam-column joints shall be equal to those provided along the critical regions of the column. Besides, at least one intermediate vertical bar shall be provided between column-corners at each side of the joint.

2.6. R.C. Walls

This chapter contains the aseismic design rules of R.C. walls or plane elements under (mainly) in-plane action-effects.

1. Design actions effects
2. Walls of DC "H" structures
3. Walls of DC "M" structures
4. Walls of DC "L" structures

2.6.1. Design actions effects

1. Analysis

2. Distinction of prevailing failure modes

2.6.1.1. Analysis

a) The acting bending moments. shear forces and axial forces on walls shall be determined from the analysis of the structure for the seismic load combination.

However, for DC "H and "M" walls and whenever a static analysis is carried out, dynamic effects should be taken into account by means of an appropriate simplified method. If a more precise method is not available, the rules given in Appendix C may be used for the estimation of the final action-effects to be taken into account in dimensioning and detailing. These rules cover theca deign envelopes for bending moments, as well as the magnification factors for shear forces.

b) The possible modifications of axial forces on walls when the seismic direction is changing, shall be appropriately taken into account.

2.6.1.2. Distinction of prevailing failure modes

a) Because of the fundamental importance of the prevailing mode of failure on the behaviour factor to be used in analysis (Clause. 2.1.4.) a prediction of the expected type of failure (flexural or shear) should be made for each wall of DC "H" and DC "M" structures. For DC "L" structures such a distinction is not needed since the behaviour factor is very close to unity.

b) The distinction between these two types of failure may be made on the basis of the ratio:

$$\upsilon = V_{Rd} / (\gamma_{Rd} * V_{MRd})$$

where VRd = the minimum, value of the design strength versus shear failure modes (diagonal compression. diagonal tension or shear sliding) as described in Clause. 2.6.2.1.

V(MRd) The shear force value at the state of design flexural failure of the critical region under consideration i.e. when the acting moment equals the design flexural strength Mrd ; this force depends on Mrd and on the structural system (e.g. for a free cantilever wall with only one horizontal force acting on its top (V(Mrd) = Mrd/hw).

More generally V(Mrd) = Mrd/ás/lw where:

αs the shear ratio at the state of failure.

γrd a global factor expressing the uncertainties :

of the models predicting the shear and flexural strengths and,

of the shear ratio value needed in order to translate Mrd into V(Mrd) values.

When more precise data are not available ărd may be taken equal to 1.40 for DC "H" structures and to 1.30 for DC "M" structures.

c) By means of the υ-ratio. the distinction of the type of failure may be made as follows for DC "H" and "M" walls:

(i) if 1.00 < u a flexural type of failure is highly probable

(ii) if 0.75 < u < 1.00 a mixed type of failure will probably occur, however with a prevailing flexural character

(iii) if 0.50 < u < 0.75 a shear type of failure is highly probable

(iv) if u < 0.50 a shear type of failure is certain

2.6.2. Walls of DC "H" structures

For the design of walls in strutures of ductility class "H" the following issues need to be resolved:

1. <u>Design strength evaluation and verification</u>
2. <u>Coupling beams</u>
3. <u>Local ductility</u>
4. <u>Specific measures</u>

2.6.2.1. Design strength evaluation and verification

1. Flexural failure

   Flexural strengths shall be evaluated and verified as for columns, under the minimum compressive axial force for the seismic load combination.

2. Web diagonal compression failure

$$V_{Sd} \leq V_{Rd2} \text{———} (2.32)$$

where

$$V_{rd2} = 0.20 * f_{cd} * b_w * l_w \text{\_in\_critical\_regions\_} (2.33)$$

and

$$V_{rd2} = 0.30 * f_{cd} * b_w * l_w \text{\_outside\_these\_regions\_} (2.34)$$

3. Web diagonal tension failure

$$V_{Sd} \leq V_{Rd3}$$

An accurate model shall be used for the evaluation of the shear resistance VRd,3 taking also into account the cyclic reversals of post yield imposed deformations. If a more precise model is not available, the following provisions shall apply:

a) When αs > 1.3, a simplified truss model may be used.

Consequently,

Horizontal bars. fully anchored at the boundary elements of the wall's cross-section. shall be provided along the height of the wall to satisfy the inequality:

$$V_{Sd} \leq \rho_h * f_{yd,h} * b_w * l_w + V_{cd} \text{———} (2.36)$$

Vertical bars, properly anchored and spliced along the height of the wall shall also be provided also the web to satisfy the inequality

$$V_{Sd} \leq \rho_v * f_{yd,h} * b_w * l_w + \min N_{sd} \text{———} (2.37)$$

b) When αs < 1.3 the following empirical expression shall be used for calculation of the required horizontal and vertical reinforcement:

$$V \leq [\rho_h * f_{yd,h} * (\alpha_{\varsigma} - 0.3) + \rho_v * f_{yd,v} * (1.3 - \alpha_{\varsigma})] * b_w * l_w + V_{cd} \text{—} (2.38)$$

(where if αs < 0.3 it shall be taken άs = 0.3).

c) The term Vcd may be approximated as follows.

For low axial load (Nsd <0.10*lw*bw*fcd)

1. In the critical region of the wall: Vcd = 0
2. Outside the potential hinge area: (Vcd = 2.5*τrd*bw*lw)

For high axial load (Nsd > 0.10*lw*bw*fcd)

1. In the critical region of the wall: (Vcd = 2.5*τrd*bw*lw)
2. Outside the potential hinge area: (Vcd = 2.5*τrd*bw*lw)

d) In every case, the minimal ñh, ñv, values foreseen in Clause. 2.6.2.4 shall be respected.

4. Shear sliding failure

$$V_{sd} \le V_{Rd,s} \text{—, where}$$
$$V_{Rd,s} = V_{dd} + V_{id} + \mu_f * \min N_{Sd}$$

$$V_{dd} = 1.5 * \sum_j A_{sj} * \sqrt{f_{cd} * f_{yd}} \quad \text{(dowel resistance of vertical bars) (2.45)}$$

$$V_{id} = 2 * \sum_i A_{si} * f_{yd} * \cos\varphi \quad \text{(shear resistance of inclined bars) (2.46)}$$

µf 1/3 a safe lower value of concrete-to-concrete friction coefficient under cyclic actions.

2.6.2.2. Coupling beams

In order to ensure a highly dissipative behaviour of beams connecting adjacent structural walls, the evaluation and verification of strength of these beams shall be carried out as follows:

a) If one of the following conditions is fulfilled, the provisions of Clause. 2.2.1.3 apply:

1. A bidiagonal cracking has little probability to occur. if a more precise rule is not available, this happens when:

$$V_{Sd} \le 6 * b * h * \tau_{rd} \text{—}(2.47)$$

2. A prevailing flexural mode of failure is secured. If a more precise rule is not available, this happens when:

$$l/h \ge 3 \text{—}(2.48)$$

b) Otherwise, all seismic actions shall be resisted by bidiagonal reinforcement. The Following conditions should be observed:

$$V_{Sd} = \frac{2}{l} * M_{Sd} \le 2 * A_{Si} * f_{yd} * \sin\alpha$$

where Asi = total area of steel bars provided in each diagonal direction

α = slope of diagonals to the horizontal.

1. This reinforcement is arranged in column-like elements; the anchorage length of these reinforcements shall be increased by 50% of the lengths prescribed in EC2.
2. Hoops shall be provided across these column-like elements in order to prevent buckling of their longitudinal bars. The provisions of Clause. 2.3.1.3 and Clause 2.3.2.3 apply; however sh < 100 mm.

In all cases, the specific measures of Clause. 2.6.2.4, as well as those of beams outside critical regions, apply for coupling beams as well.

2.6.2.3. Local ductility

a) At the critical regions as defined in Clause. 2.6.2.3 near the base of walls, a conventional curvature ductility factor (CCDF) equal to

$$\mu_{1/r} = 1.2 * q^2 \_\_for\_\_uncoupled\_\_walls\_\_(2.50a)$$

$$\mu_{1/r} = 1.0 * q^2 \_\_for\_\_coupled\_\_walls\_\_(2.50b)$$

shall be secured.

In Equ.2.50, "q" denotes the value of behaviour factor which has been linearly used in the analysis, after taking into account the consequences of the relevant failure mode distinction (as stated in Clause. 2.6.2.1).

If a more precise method is not used, the implementation of Equ. 2.50 may be made by means of confining reinforcement defined as follows:

For normal cases of free-edge walls, when a "single hoop" (or cross-ties) pattern is used, the volumetric mechanical ratio of the required confining reinforcement arranged every bo/4, may be taken from the following expression:

$$\omega_{wd} = 0.9 * \sqrt[3]{\mu_{1/r}} * (0.01 * \mu_{1/r} + 0.15 * \frac{A_c}{A_S} + V_d - 0.4) \_\_(2.51)$$

$$\omega_{wd} \geq 0.20$$

where

$$\omega_{wd} = \frac{Volume\_\_of\_\_stirupps\_\_and\_\_cross - ties}{Volume\_\_of\_\_confined\_\_concrete} * \frac{f_{yd}}{f_{cd}}$$

μ1/r	the curvature ductility demand (Equ. 2.50)
Vd	(max Nsd)/bw*lw*fcd
Ac	concrete gross area along the confined length lc
Ao	core concrete area along the confined length lc
lc	confined length as defined in Clause. 2.6.2.3.

b) This confinement shall be extended vertically along the critical region as defined in Clause. 2.6.2.3.c, and horizontally up to the point where, under cyclic loading. unconfined concrete may be spalled due to large strain. If more precise data are not available. this critical strain value may be taken equal to 0.15% : the relevant loading situation should be under Msd and max Nsd.

c) It shall be considered that a potential area for plastic hinge formation. (critical region). extends from the wall's base up to a level greater than:

-The wall's length

-The storey's height. or

-1/6 of the walls height.

All requirements and detailing rules for columns' reinforcements (longitudinal and transversal) apply also to the confined boundary areas of walls.

2.6.2.4. Specific measures

The criterion of Clause. 2.1.2.1.3 may be secured as follows:

a) Walls shall be secured against premature web shear cracking: therefore. a minimum amount of web reinforcements shall be provided equal to:

$$\rho_{h,min} = \rho_{v,min} = 0.25\% \_\_(2.52)$$

Web reinforcements shall form two "equivalent" orthogonal grids (of bars with the same bond characteristics), one on each side of the wall's web; grids shall be connected by adequate and properly spaced cross-ties.

Anchoring and splicing of web reinforcements follow the provisions of EC2.

b) To counterbalance the unfavourable effects and the uncertainties in case of unpredicted cracking along cold joints, a minimum amount of well anchored reinforcement shall be provided across expected cold joints. The reconstitution of the resistance o' uncracked concrete against shearing is sought:

$$\rho_{min} = \sqrt{f_{cd} * f_{yd}} + \mu_f * (\rho_{min} * f_{yd} + \frac{N_{Sd}}{b_w * l_w}) \geq 1.3 * f_{ct,0.05} \_\_(2.53)$$

where

µf friction coefficient, may be taken equal to unity

Nsd the minimum compressive force acting on the wall.

Therefore:

$$\rho_{min} \geq (1.3 * f_{ct,0.05} - \frac{N_{Sd}}{b_w * l_w}) / f_{yd} / (1 + \frac{f_{cd}}{f_{yd}}) \geq 0.25\% \_\_(2.54)$$

c) Web thickness should not be less than 150 mm or hs/20.

d) In order to avoid unpredicted lateral instability of the confined elements of walls (Clause. 2.6.2.3), their thickness should observe the following rules:

$$if\_\_l_c \geq 2 * b_w \_\_then\_\_b_w \geq 200\_\_mm$$
$$if\_\_l_c \geq 0.2 * l_w \_\_then\_\_b_w \geq h_s / 10$$

$$if\_\_l_c < 2 * b_w \_\_then\_\_b_w \geq 200\_\_mm$$
$$if\_\_l_c < 0.2 * l_w \_\_then\_\_b_w \geq h_s / 15$$

(iii) In case the most compressed edge of the well is connected to an adequate transversal flange (minbf = hs/15. min if = hs/5), then if lc < 3bw

$$it\_\_shall\_\_be\_\_b_w \geq \max\{150mm, h_s / 20\}$$

a) A minimal confined boundary element should be provided at free edges (if not connected to adequate flanks), containing hoops or cross-ties of no less than in diameter 6mm every bo/4.

b) The transfer of cyclic diaphragm actions from the floor to the walls shall be appropriately checked.

c) Random openings not regularly arranged to form coupled walls shall be avoided, unless their influence is either insignificant or accounted for by means of an appropriate analysis.

2.6.3. Walls of DC "M" structures

For the design of walls in strutures of ductility class "M" the following issues need to be resolved:

1. Design strength evaluation and verification
2. Coupling beams
3. Local ductility
4. Specific measures

2.6.3.1. Design strength evaluation and verification

All provisions of Clause. 2.6.2.1. apply.

2.6.3.2. Coupling beams

All provisions of Clause. 2.6.2.2 apply. However, if the conditions of Clause. 2.6.2.2.a do not occur, instead of the structural model of the bidiagonal column-like elements, other models may be used if adequately proved to ensure a comparable level of energy dissipation capacity without a substantial force-response degradation.

2.6.3.3. Local ductility

All provisions of Clause. 2.6.2.3 apply: however $\omega wd > 0.15$. It is reminded that the confining reinforcement of the boundary elements is arranged every bo /3.

2.6.3.4. Specific measures

All provisions of Clause. 2.6.2.4 apply with the following modification:

Web reinforcements should not be arranged at distances more than 25 lengths of diameter or 250 mm.

2.6.4. Walls of DC "L" structures

All provisions of EC2 relevant to structural walls apply. Nevertheless, the following additional rule should be respected in order to ensure a conventional curvature ductility factor compatible with the q-values foreseen in Table V, Clause. 2.1.4:

For normal cases of free-edge walls, when a "single hoop" (or cross-ties) pattern is used, the volumetric mechanical ratio of the required confining reinforcement arranged every bo/2 shall be:

$$\omega_{wd} = \min\{1.45 * (0.10 * \frac{A_c}{A_s} + v_d - 0.37), 0.10\}$$

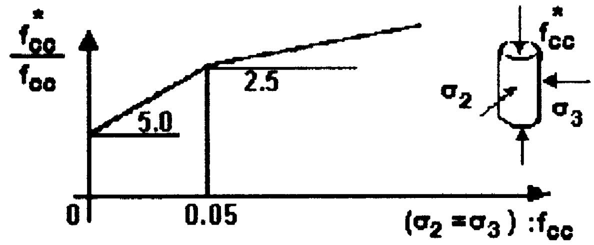
Appendix "A"

Practical expressions of characteristics of confined concrete

Based on the finding (see Addendum) that the average effective confining pressure is given by the expression:

$$\mathrm{eff}\frac{\sigma_2 = \sigma_3}{f_{cc}} = \frac{1}{2} * \alpha_n * \alpha_s * \omega_w$$

and that the increase of concrete strength (due to the triaxial effect) is given by the diagram here below:



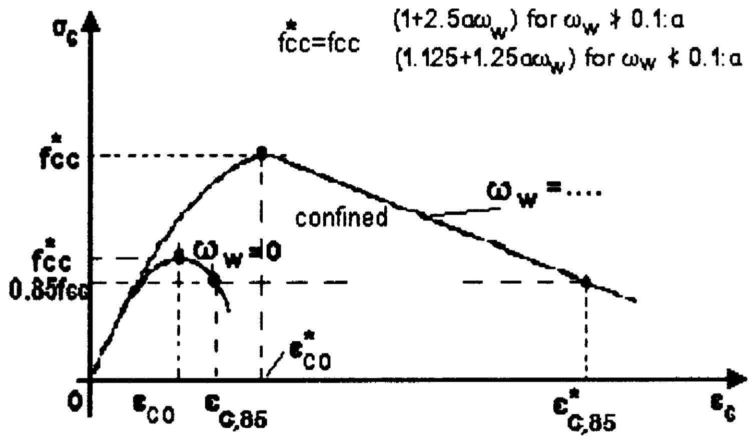
the following characteristics of the modified constitutive law of the confined concrete may be used in a closed form:

$$f^*_{cc} = f_{cc} * (1.0 + 2.5 * \alpha * \omega_w)\_for\_\frac{\sigma}{f_{cc}} \leq 0.05$$

$$f^*_{cc} = f_{cc} * (1.125 + 1.25 * \alpha * \omega_w)\_for\_\frac{\sigma}{f_{cc}} \geq 0.05$$

$$\varepsilon^*_{co} = \varepsilon_{co} * (f^*_{cc} / f_{cc})^2$$

$$\varepsilon^*_{c,85} = \varepsilon_{c,85} + 0.1 * \alpha * \omega_w\_(s.\_also\_Fig.\_1)$$



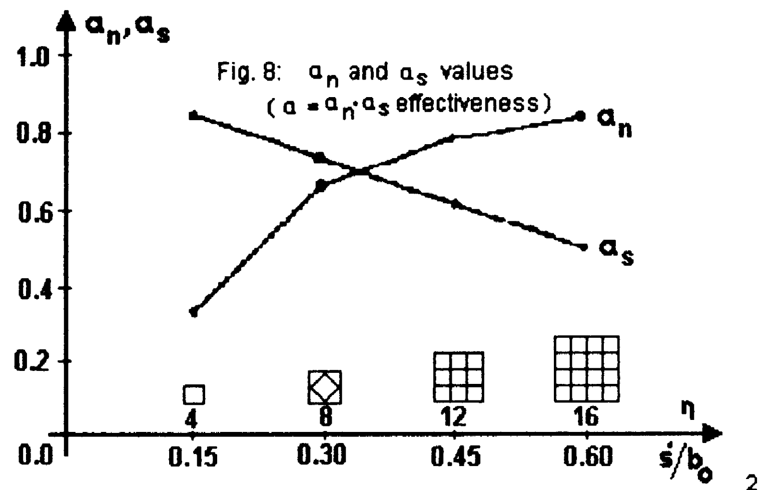


Fig. 8: a_n and a_s values ($a = a_n \cdot a_s$ effectiveness)

Note: for circular columns and circular hoops $a_n = 1$, $a_s = (1-s/2b_0)^2$
for circular columns and spiral reinf. $a_n = 1$, $a_s = (1-s/2b_0)$

Besides these conditions, the problems of irregularity should be considered.

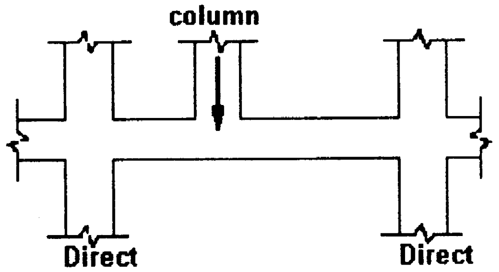
If more precise data are not provided, the effect of vertical component could be taken into account as follows:

Design axial load of cut-off column, to be taken into account in beam's strengths verifications. shall be multiplied by a factor equal to:

$$(1 \pm q * \varepsilon)$$

where "e" denotes the seismic coefficient of the beam' s floor level and "q" is the building's behaviour factor.

In calculating beam's design action-effects, the load +-Nsd*q*å be considered as a live load, with a Ø2 factor equal to 1.00.



Appendix "B"

Specific rules for cut-off vertical elements (resting-on beams)

1). Vertical elements (walls or columns) resting on slabs are not permitted in seismic zones.

2). Cut-off walls resting on beams are not permitted.

3). Cut-off columns resting on beam's of DC "H" are not permitted.

4). Cut-off columns resting on beams of DC "M" and "L" could be permitted, under the following conditions:

i) Eccentricity of the column's axis relative to the beam's axis shall be zero (Clause. 2.2.1.2).

ii) Beams shall be resting on at least two direct supports, on walls or columns.

iii) Simultaneously acting vertical component of seismic action shall be taken into account.

iv) Flexural resistance

It is evaluated as foreseen in Clause. 2.2.1.2 and Clause. 2.3.1.2

A minimum amount of up-side reinforcement shall be provided in the critical region of potential hinge formation.

v) Shear resistance

It is evaluated as foreseen in Clause. 2.2.1.2 and Clause. 2.3.1.2

In addition to the minimum amount of stirrups foreseen for critical regions, a minimum amount of bidiagonal reinforcement shall be provided in the critical regions of potential hinge formation.

Appendix "C"

Specific rules for the estimation of the final design action effects of R.C. walls

a) Bending moments' design envelopes

The design bending moment diagram along the height of the wall shall be that given by a linear envelope of the calculated bending moment diagram (obtained from the structural analysis), vertically displaced by a distance equal to the length o! the wall.

b) Shear forces' magnification factors

The design shear forces along the height of the wall shall be those given by the calculated shear forces (obtained from the structural analysis) increased by the magnification factor "ω".

i. For buildings with n < 5

$$\omega = 0.9 * 0.1 * n \_ (\omega = 1.0 \leftrightarrow 1.4)$$

ii. For buildings with n > 5

$$\omega = 1.2 * 0.04 * n \_ (\omega = 1.4 \leftrightarrow 1.8)$$

where n: number of storeys, n < 15

Appendix "D"

Dimensionig of R.C. walls for flexural strength and ductility

An analytical method is presented here below for the design of R.C slender walls' (of DC "H" and "M") base cross-section for a given q-factor.

It is assumed that the behaviour factor q of the building (essentially wall system) and the overall deflection ductility factor of the walls may be interrelated as follows:

$$\sqrt{2 * \mu_d - 1} < q < \mu_d \_ or \_ q < \mu_d < \frac{1}{2} * (q^2 + 1)$$

If this is so the following formula could be used for the estimation of the required curvature ductility factor:

$$\mu_{1/r} = 1 + \frac{(\mu_d - 1)}{3 * \lambda_{pi} * (1 - 0.5 * \lambda_{pi})}$$

where

lpi lpi/hw

lpi Plastic hinge length.

Thus, for a given behaviour factor q a minimum curvature ductility should be secured:

$$\mu_{1/r} = (\frac{1}{r})_u / (\frac{1}{r})_y \cong f(q, \lambda_{pi})$$

In other words:

$$(\frac{1}{r})_u = f(q, \lambda_{pi}) * (\frac{1}{r})_y$$

where, approximately:

(1/r)y=0.8*Mp/3/K

(k' denotes a second stiffness of the wall-base up to its yield point)

The strains at the critical region of the wall should satisfy the condition:

$$\varepsilon_{cu} + \varepsilon_{su} = (\frac{l_w}{r})_u \cong f(q, \lambda_{\rho i}) * (\frac{l_w}{r})_y = \text{constant}$$

(All possible rotated section lines satisfying this condition are parallel).

If the approximations proposed along this Appendix are adopted, the following formulation of this condition may be used:

$$\varepsilon_{cu} + \varepsilon_{su} \cong f(q, \lambda_{\rho i}) * 0.8 * M_R * l_w / 3 / K$$

In addition, the equilibrium equations should be respected:

$$N_{sd} = F_c - F_s = F_{cc} + \sum A_{si,2} * \sigma_{si,2} - \sum A_{si,1} * \sigma_{si,1}$$

$$M_{sd} + N_{sd} * l_w / 2 = F_c * y_c - F_s * y_s$$

Wherever the computed concrete strain exceeds the value.

$$\varepsilon_{c,cr} \cong (1/2 \approx 1/3) * \varepsilon_{cu,conv} \cong 0.15\%$$

confinement (secured by adequate hoops of mechanical volumetric percentage "ùw") should be provided.

For the satisfaction of these equilibrium conditions, the stress/strain diagram of confined concrete may be taken in an approximate way as s function of the volumetric mechanical ratio ùw of the confining reinforcements.

The solution of the set of Eqs.(3) to (5) may be found by trial and error leading to the evaluation of the amount and extent of the required confining reinforcement (ùw) and of the confined length (lc).

SPECIFIC RULES FOR STEEL STRUCTURES

3.1 General

0. Symbols
1. Scope
2. Earthquake-resistant structures
3. Strength verifications

3.1.0 Symbols:

A+,A	horizontal projections of the areas of the tension diagonals
At,Ac	areas of the diagonals in tension or compression
M	bending moment in the beam or column due to the action effects
Mca, Mcb	moment resistances of the joints
Mco	bending moment due to vertical loads for the seismic loading condition
Mcs	bending moment due to the seismic loads
Mpa, Npb	moment resistances of the beam
Mpd, Npd	ultimate resistances according to Chapter 5 of Eurocode 3
Mra	min{Mpa,Mca}
Mrb	min{Mpb,Mcb}
N	axial force in the beam or column due to the action effects
Nd	resistance of the beam or the column to the axial force N
Rd	resistance of the connection according to Chapter 6 of Eurocode 3
Rfy	yielding resistance of the connected part
V	shear force in the beam or column due to the action effects
Vm	shear force due to the resisting end moments Mra and Mrb of the beam
Vo	action effect due to vertical load in the seismic situation
Vpd	shear resistance of the beam according to Chapter 5 of Eurocode 3
b/t	width-thickness ratio
fy	yield strength
fyri	actual yield strength of the steel component i on site
fyi	yield strength of the steel component i considered in the design

q	behaviour factor
ri	ratio fyri/fyi
α	magnification factor defined in Clause 3.5.5.2
α1	multiplier for horizontal seismic actions to reach the limit state in the first hinge
αu	multiplier for horizontal seismic actions to reach the collapse load
γm	partial safety factor to the resistances for fundamental load combinations
ε	(235/fy)^0.5
λ	slenderness

3.1.1 Scope

Steel structures in seismic regions shall be designed in accordance with Eurocode 3.

Those parts of the structure which are intended to resist the effects of seismic actions, henceforth defined as earthquake-resistant, shall satisfy the rules laid out in this chapter.

Structural or non-structural elements fixed to earthquake-resistant structures shall comply with the rules in chapter 8. of this part.

3.1.2 Earthquake-resistant structures

Earthquake-resistant structures may be designed according to the following concepts:

a) Concept of non-dissipative structures

The action effects shall be calculated on the base of an elastic analysis without taking account of non-linear material behaviour. This implies using a behaviour factor q = 1.

For these structures the strength of the members and of the connections to resist the design seismic loading shall be evaluated in accordance with the rules for elastic or plactic resistances in Eurocode 3 without any ductility requirements.

b) Concept of dissipative structures

They are designed in such a way that during an earthquake some of their parts, which will henceforth be refered to as dissipative zones, will move out of the elastic range in order to dissipate energy by means of ductile hysteretic behaviour. The energy dissipation of these parts is taken into account by a behaviour factor q > 1.

3.1.3 Strength verifications

The combination of actions shall be taken from Clause 6.2 of part 1.1.

The γm-values specified in Eurocode 3 for fundamental load combinations shall be applied also to the resistances for seismic loading.

3.2 Materials

Steels, welding and bolts shall conform to the requirements specified in Chapter 3 of Eurocode 3.

In dissipative zones the following materials shall be used :

1) general structural steels according to EN 10025.

2) for bolted connections preferably high strength bolts in category 8.8 or 10.9 tightened as prescribed for friction joints. Bolts in category 12.9 in tension are not allowed.

3) the maximum value of the yield strength and tensile strength of the steel to be used shall be specified.

3.3 Structural types

1 Non-dissipative earthquake-resistant structures

In case of non-dissipative earthquake-resistant structures no distinction is needed for the type of he structure in view of the behaviour under earthquakes because ductility effects are neglected.

2 Dissipative earthquake-resistant structures

Dissipative structures are classified according to their seismic behaviour into the following types (see fig. 3.3.2.1)

1. Frame structures which resist horizontal forces in an essentially flexural manner. In these structures the dissipative zones are mainly located in plastic hinges near the beam-column joints and energy can be dissipated by means of cyclic bending.

2. Concentric truss bracing in which horizontal forces are mainly resisted by bars subjected to axial forces. In these structures the dissipative zones are mainly located in the tensile diagonals. Dissipative concentric truss bracings are subdivided in

   2.1. Diagonal bracings, in which the horizontal external loads can be resisted by the tension diagonals only, neglecting the compression diagonals.

   2.2. V-bracings, in which the horizontal external loads can be resisted by considering both tension and compression diagonals. The intersection point of these diagonals lies on the horizontal member which shall be continuous.

   2.3. K-bracings, in which the diagonal intersection lies on the column are not considered as dissipative because this would require the co-operation of the column to the yielding mechanism.

3. Eccentric truss bracing in which horizontal forces are mainly resisted by axially loaded bars and the excentricity of the layout is such that energy shall be dissipated in the beams by means of either cyclic bending or cyclic shearing. Only those bracings in which yielding due to bending or shear of the beams or of parts of them precedes the attainment of the limit strengths of the tension or compression bars can be associated to this group.

4. Cantilever structures or inverted pendulum structures acting essentially as beam-columns. In these structures dissipative zones are mainly located at the base.

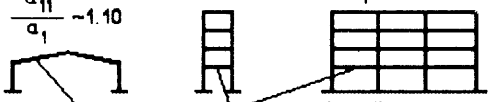
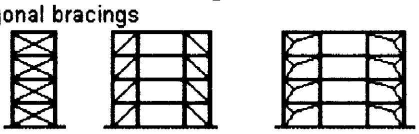
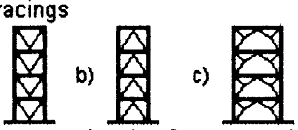
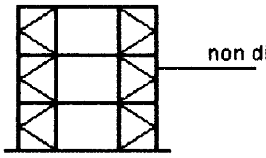
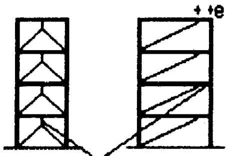
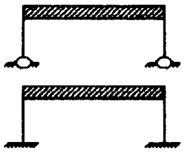
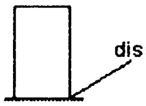
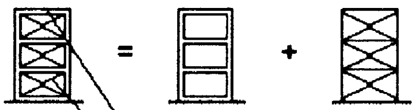
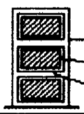
5. Structures with reinforced concrete cores or walls, in which horizontal forces are mainly resisted by these cores or walls shall be designed according to Chapter 2 of this part.

6. Dual structures, in which horizontal forces are resisted by both steel-frames and steel-bracings acting in the same plane.

7. Mixed structures in steel and reinforced concrete, in which horizontal forces are resisted by steel frames with concrete infill elements. The concrete elements shall be designed according to Chapter 2 of this part, the joints shall be designed according to Chapter 4 of this part.

3.4 Behaviour factors q

The behaviour factor q introduced in clause 4.3 of Part 1.1 of this Eurocode to account for energy dissipation capacity, takes the values in fig. 3.3.2, provided the regularity requirements laid down in part 1.2 and the design and detailing rules that are included in clause 3.5 of this part are observed.

	high regularity	medium regularity
1. Frame structures $\frac{\alpha_{11}}{\alpha_1} \sim 1.10$ $\quad \frac{\alpha_{11}}{\alpha_1} \sim 1.20$  diss. zones $\quad$ diss. zones = bending zones in the beams	$q=5\,\frac{\alpha_{11}}{\alpha_1}^*$	$q=4\,\frac{\alpha_{11}}{\alpha_1}^*$
2. Concentric truss bracings Diagonal bracings  diss. zones = tension diagonals only	q=4	q=3
V - bracings a) b) c)  diss. zones = tension & compression diagonals	q=2	q=1.5
K - bracings  non dissipative	q=1	q=1
3. Eccentric truss bracings  $\frac{\alpha_{11}}{\alpha_1} \sim 1.10$ diss. zones = bending or shear zones	$q=5\,\frac{\alpha_{11}}{\alpha_1}^*$	$q=4\,\frac{\alpha_{11}}{\alpha_1}^*$
4. Cantilever structures  diss. zones in the columns  Restrictions. $\lambda \leq 1.5$; $\Theta \leq 0.2$ (see clause 3.5.7.1)	q=2	q=1.5
5. Cores or walls in reinforced concrete  dissipative zones	chapter 7	chapter 7
6. Dual structures  frames with diss. bending zones bracings with diss. tension zones	$q=5\,\frac{\alpha_{11}}{\alpha_1}^*$	$q=4\,\frac{\alpha_{11}}{\alpha_1}^*$
7. Mixed structures made from steel frames with reinforced concrete infills  frames with diss. bending zones diss. ahear walls diss. joints	q=2	q=1.5

The following parameters are used in the previus figure:

α_1 multiplier of the horizontal seismic actions, by keeping constant the other design loads, which corresponds to the point, where the most strained cross section reaches its limit state.

α_u multiplier of the horizontal seismic actions, by keeping constant the other design loads, which corresponds to the point, where the structure reaches the Maximum load bearing capacity taking into

account of bending moment redistribution's due to the formation of plastic hinges.

In case appropriate calculations are not performed in order to evaluate the collapse-multiplier α_u, the approximate values of the ratio α_u/α_1 in fig. 3.3.2 may be used.

In the case of highly regular buildings to be built in regions defined by national authorities and having earthquake resistant structures made from rolled sections and conforming to the types listed in fig. 3.3.2 (the K-bracing excepted) a behaviour factor $q = 1.50$ may be adopted without taking account of the limitations given in clause 3.2 and the detailing rules given in clause 3.5. The general safety requirement set forth in Part 1.1 of this Eurocode may then be deemed to be satisfied by the strength assessments according to Chapter 5 and Chapter 6 of Eurocode 3.

3.5 Design criteria and detailing rules for dissipative structures

1. Design criteria
2. Detailing rules for structural elements
3. Diaphragms and horizontal bracing
4. Frames
5. Concentric truss bracings
6. Eccentric truss bracing
7. Cantilever structures or inverted pendulum structures
8. Dual structures
9. Mixed structures made from steel frames with reinforced concrete infill

3.5.1 Design criteria

This clause contains the design criteria for dissipative earthquake-resistant structures, for which behaviour factors q according to fig. 3.3.2 are taken into account.

Structural systems with dissipative zones shall be designed such that these zones develop mainly in those parts of the structure, where yielding or local buckling or other phenomena due to hysteretical behaviour do not affect the overall stability of the structure.

Structural parts of dissipative zones shall have adequate strength and ductility. The strength shall be verified according to Chapter 5 and 6 of Eurocode 3.The ductility requirement is deemed to be satisfied, if the detailing rules specified hereafter are met.

Non-dissipative parts of dissipative earthquake resistant structures and the connections of the dissipative parts to the rest of the structure shall have sufficient over strength to allow cyclic yielding of the dissipative parts to develop.

If the detailing rules that follow are infringed the earthquake-resistant structure shall be designed as a non-dissipative structure.

3.5.2 Detailing rules for structural elements

1 Parts in compression

Sufficient local ductility of members or parts of members in compression shall be assured by restricting the width-thickness ratio b/t.

In dissipative zones of structures calculated with $q < 2$ the width-thickness ratios b/t shall conform to the limits for class 3-sections in Eurocode 3.

In case behaviour factors $q > 2$ are chosen, the width-thickness ratios b/t shall comply with the conditions in table 3.5.1 and table 3.5.2.

2 Connections

Connections in dissipative zones shall have:

sufficient overstrenqth to allow for yielding of the connected parts. For these overstrength verifications the upper value of the yield strength of the connected parts shall be taken.

Connections of dissipative parts made by means of butt-welds or full penetration groove welds are deemed to satisfy the overstrength criterion.

For fillet weld connections or bolted connections the following requirement shall be met

$$R_d \square 1.20 \, {}^*R_{fy}$$

where

Rd Resistance of the connection according to Chapter 6 of Eurocode 3

Rfy Yielding resistance of the connected part.

Table 3.5.2 Limit b/t ratios of compressed parts of cross-sections for different cross-sections classes

Cross section and boundary condition	Stress distribution (compression positive)	[illegible]		
Rectangular hollow section	compression	33 [illegible]	[illegible]	[illegible]
Tubular section	compression	$50\ \varepsilon^2$	$70\ \varepsilon^2$	$85\ \varepsilon^2$
Webs of I-profiles, Webs of flanges of welded sections	plastic distribution / elastic distribution	66 [illegible]	[illegible]	90 [illegible]
	compression	33 [illegible]	[illegible]	41 [illegible]
	combined bending and compression; plastic distribution / elastic distribution	[illegible]	[illegible]	[illegible]
Outstanding flanges of welded box sect. or flanges of T-profiles	compression	9 [illegible]	10 [illegible]	12 [illegible]
	combined bending and compression	[illegible]	[illegible]	[illegible]
	combined bending and compression	[illegible]	[illegible]	[illegible]
Flanges of I-profiles	compression	[illegible]	[illegible]	[illegible]

General

$\varepsilon = \sqrt{235/f_y}$

f_y	235	275	355
ε	1	0.92	0.81

3 Parts in tension

The overstrength condition for connections may be overlooked in case the connections are designed such, that they contribute significantly to energy dissipation to achieve the chosen q-factor.

The effectiveness of such connection devices and their strength in view of cyclic behaviour shall be proved by tests, to the satisfaction of national authorities.

For bolted shear connections the bearing failure shall precede the shear failure.

For tension-members or parts of members in tension the ductility-requirement in clause 5.4.3.2 in Eurocode 3 shall be met.

3.5.3 Diaphragms and horizontal bracing

The horizontal diaphragms and bracings shall be able to transmit with sufficient overstrength the earthquake forces to the various earthquake-resistant elements which are connected by them.

This condition is assumed to be fulfilled if for the relevant verifications the forces obtained from the analysis are multiplied by a magnification factor α = 1.50.

The following minimum detailing rules shall be satisfied for diaphragms in reinforced concrete:

1. The horizontal diaphragms shall be reinforced in two directions and the reinforcements shall be anchored to the perimeter beams.
2. When the diaphragms consist of parallel-ribbed floors, an additional reinforcement shall be placed at right angles to it within the upper layer of concrete. This reinforcement shall consist of no less than 4 bars of 8 mm diameter every meter. It may also be housed in special cross way ribbings located at an interaxis of no more than 2,5 m.
3. Prefabricated plates may be used, provided each has reinforcements in two directions at right angles to each other and they are connected to the support beams in such a manner that the whole produces a truss system in the horizontal plane.

3.5.4 Frames

1 Specific criteria

Frames shall be designed so that plastic hinges form in the beams and not in the columns. This criterion is waived at the base of the frame, at the top floor of multi-storey buildings and for one storey buildings.

The beam to column joints shall have adequate over strength to allow the plastic hinges to be formed in the beams.

The required hinge formation pattern shall be achieved by observing the following rules.

2 Beams

Beams shall be verified as having sufficient safety against lateral or lateral torsional buckling failure according to Chapter 5.5.2 of Eurocode 3 assuming however the formation of a plastic moment at one end of the beam.

For plastic hinges it shall be checked that the full plastic moment resistance and rotation capacity is not decreased by compression and shear forces. To that end the following inequalities shall be verified at the location where the formation of hinges is expected.

$$\frac{M}{M_{pd}} \le 1 \qquad (3.2)$$

$$\frac{N}{N_{pd}} \le \frac{1}{10} \qquad (3.3)$$

$$\frac{V_g + V_M}{V_{pd}} \le \frac{1}{3} \qquad (3.4)$$

where

N,M are the action effects taking account of the behaviour factor

Npd,M pd are the ultimate resistances according to Chapter 5 of Eurocode 3

Vo is the shear force due to vertical loads

Vm = (Mra + Mrb)/l is the shear force due to the resisting moments Mra and Mrb of the beam at its extremities A and B with

Mrad = min{Mpa,Mca}

Mrbd = min{Mpb,Mcb}

where

Mpad, Mpbd are the moment resistances of the beam

Mcad, Mcba are the moment resistances of the joints (see clause 3.5.2.2)

l is the distance between A and B

Vpd is the shear resistance of the beam according to Chapter 5 of Eurocode 3

3 Columns

For the verification of columns a "worst case" combination of the axial force N and the bending moments Mx and My shall be assumed.

The design values for the bending moments shall not be lesser than the sum of the resisting moments Mr of the beams connected to the column as determined in clause 3.5.4.2.

The transfer of the forces due to the end moments of the beams to the column shall comply with the design rules in Chapter 6 of Eurocode 3.

The strength verification of the columns shall be made according to Chapter 5 of Eurocode 3.

At the base of the frame the design bending moments for the connection of the column to the foundations shall be taken from

$$M = M_{co} + \alpha * M_{cs} \quad \text{—(3.5)}$$

where

Mco is the bending moment due to vertical actions

Mcs is the bending moment due to the seismic actions

α 1.2 except for the top floor of multi-storey frames, where $\alpha = 1.0$.

The shear force in a column shall be limited to

$$\frac{V}{V_{pd}} \leq \frac{1}{3} \quad \text{—(3.6)}$$

except for the framed web panels connected to the beams, where (V/Vpd) < 1 is permitted.

A joint in the column shall be designed such that its strength exceeds the resistance of the connected parts.

4 Beam to column joints

The joints of the beams to the columns shall be designed in view of sufficient over strength (see clause 3.5.2.2) taking into account the moment resistance Mpd and the shear force (Vo + Vm) evaluated in clause 3.5.4.2.

3.5.5 Concentric truss bracings

1 Specific criteria

Concentric bracings shall be designed, so that yielding of the diagonals in tension will take place before yielding or buckling of the beams or the columns and before failure of the connections.

The diagonal elements of bracing Qs shall be placed in such a way that the structure exhibits similar load-deflection characteristic at each floor and in every braced direction under load reversals. To this end the following rule shall be met

$$\frac{|A^+ - A^-|}{A^+ + A^-} \leq 0.05 \_\_(3.7)$$

where A+ and A- are the horizontal projections of the Areas of the tension diagonals, when the horizontal actions have a positive or negative direction respectively.

2 Diagonal members

The slenderness ë as defined in Chapter 5.5.12 of Eurocode 3 shall be limited to

$$\bar{\lambda} \leq 1.5 \_\_(3.8)$$

The tension force shall be limited to the yield resistance

$$\frac{N}{N_{pd}} \leq 1.0 \_\_(3.9)$$

In V-bracings the compressive diagonals shall be designed for the compression strength according to Chapter 5 of Eurocode 3.

The connections of the diagonals to any members shall fulfill the over strength condition

$$R_d \geq 1.20 * N_{pdi} \text{---}(3.10)$$

3 Beams and columns

Beams and columns with axial forces shall meet the following minimum strength requirement

$$N_d(M) \geq \alpha * 1.20 * N \_\_(3.11)$$

where

N	axial force in the beam or column under vertical loads and seismic action.
α	min{Npdi/Ni} which is the minimum ratio of the resistances Npdi and the forces Ni for all diagonals i in one bracing.
Nd	buckling resistance of the beam or the column to the axial force N according to Eurocode 3

In V-bracings (case b and c in fig. 3.3.2) the beam shall be designed to resist all bending actions without considering the intermediate support given by the diagonals.

The foundation base connections shall fulfill the minimum strength requirement as for beams and columns.

3.5.6 Eccentric truss bracing

1 Specific criteria

Eccentric truss bracings shall be designed so that beams are able to dissipate seismic energy by the formation either of plastic bending or plastic shear mechanisms.

The rules given hereinafter are intended to ensure that yielding in the plastic hinges or shear panels of the beams will take place prior to the yielding or failure elsewhere.

2 Beams

For beams which dissipate energy by plastic hinges the rules defined in clause 3.5.4.2 shall be observed.

For beams which dissipate energy by a plastic shear mechanism the following rules shall be met:

$$\frac{V}{V_{pd}} \leq 1.0 \_\_(3.12)$$

$$\frac{M}{M_{pd}} \leq 0.70 \_\_(3.13)$$

$$\frac{N}{N_{pd}} \leq 0.10 \_\_(3.14)$$

3 Columns and diagonal members

The columns and diagonal members shall be verified considering a "worst case" combination of the axial force and any bending moments:

$$N_d(M,V) \geq \alpha * 1.20 * N \_\_(3.15)$$

where

Na,Nd resistance of the column or diagonal member according to Eurocode 3

N compressive force effect in the column or diagonal member

and α=min{Vpdi/Vi,Mpdi/Mi} is the minimum ratio of the yield resistances Vpdi or Mpdi in the plastic zone of the beam and the force Vi or the moment Mi for all beams i in one bracing system.

3.5.7 Cantilever structures or inverted pendulum structures

1 Specific criteria

The structure shall be designed according to the following condition:

slenderness < 1.5

The following limitation of Θ according to clause 4.1.1.3 in part 1.2 shall be observed for the column

$$\Theta \leq 0.20 \_\_(3.17)$$

The column shall be designed taking account of P-Δ effects if appropriate.

In cantilever structures the columns and their bases shall be verified.

2 Columns and column bases

The base connection of the column to the foundation shall be verified taking the action effect

$$M = 1.20 * M_{pd} * (1 - \frac{N}{N_{pd}}) \_\_(3.18)$$

with N as defined above

3.5.8 Dual structures

Whenever both steel frames and steel bracings are present and acting in the same direction, the horizontal forces may be distributed among them according to their elastic stiffness.

The frames and bracings shall conform to the rules in the clauses 3.5.4 and 3.5.5.

3.5.9 Mixed structures made from steel frames with reinforced concrete infill

When the horizontal forces are resisted by steel frames with infilled elements made from reinforced concrete, the following rules shall be applied.

The steel frames and the bracing structure shall be verified according to the rules in this Chapter while the shear panels shall be designed according to the provision given for walls in Chapter 2. The joints shall be dimensioned according to Chapter 4.

3.6 Specific control measures

With regard to general rules for the control, the construction and use Chapter 2 of part 1.1 is referred to.

In addition to these rules the following specific requirements shall be met:

The drawings made for fabrication and erection shall indicate the details of connections, sizes and qualities of bolts and welds as well as the steel grades of the members and the allowable maximum yield strength fy in the dissipative zones.

It shall be controlled in the different phases of the fabrication and construction that

i) the specified maximum yield strength fy of the steel material is not exceeded by more than 10%.

ii) the distribution of the yield strength throughout the structure does not substantially differ from the distribution assumed in the design. This condition is satisfied, when the population of the ratios ri = (fyri / fyi) of the actual yield strength fyri of the steel component i on site to the yield strength fyi of the steel component i considered in the design is such, that (max ri - min ri) < 0.2.

iii) no change of the structure involving an increment or decrement in the stiffness or in the strength of more than 10% of the values assumed in the design may occur.

Whenever one of the above criteria is not fulfilled new computations of the structure and of its details shall be made to demonstrate its efficiency.

The control of the tightening of the bolts and of the quality of the welds shall follow the rules laid down in Chapter 7 of Eurocode 3.

SPECIFIC RULES FOR COMPOSITE STRUCTURES

4.1 General

Symbols:

b/t	width-thickness ratio
q	behaviour factor
γm	partial safety factor to the resistance for fundamental load combinations

1 Scope

Composite structures are structures composed of composite elements as defined in Eurocode 4.

Composite structures in seismic regions shall be designed in accordance with Eurocode 4..

Those parts of the structure which are intended to resist the effects of seismic actions, henceforth defined as earthquake-resistant, shall satisfy the rules laid down in this chapter.

Structural or non structural elements fixed to earthquake-resistant structures shall comply with the rules in chapter 8.

2 Earthquake-resistant structures

Earthquake-resistant structures may be designed according to the following concepts:

a) Concept of non-dissipative structures

The action effects shall be calculated on the base of an elastic analysis without taking into account non-linear material behaviour. This implies a behaviour factor q = 1.

For these structures the strength of the members and of the connections to resist the design seismic loading shall be evaluated in accordance with the rules for the resistances defined in Eurocode 4 without any ductility requirements.

b) Concept of dissipative structures

They are designed in such a way that during an earth-quake some of their parts, which will henceforth be referred to as dissipative zones, will move out of the elastic range in order to dissipate energy by means of ductile hysteretic behaviour. The energy dissipation of these parts is taken into account by a behaviour factor q > 1.

3 Strength verifications

The load combinations shall be taken from Clause 6.2. of part 1.1

The γm-values specified in Eurocode 4 for fundamental load combinations shall be applied.

4.2 Materials

The materials shall conform to the requirements specified in Eurocode 4 with the restrictions given in chapter 2 of this part for reinforced concrete and in chapter 3 for steel.

4.3 Structural types

1 Non-dissipative earthquake-resistant structures

In case of non-dissipative earthquake-resistant structures no distinction is needed for the type of the structure in view of the behaviour under earthquakes because ductility effects are neglected.

2 Dissipative earthquake-resistant structures

Dissipative structures may be designed in the following two alternative ways:

Approach 1:

Sufficient strength and ductility is supplied by the steel parts only and the strength of the reinforced concrete parts is neglected due to the fact that failure of the concrete parts in compression or shear is expected before the steel parts yield.

However, when the steel parts are embedded in concrete parts, reducing the risks of local buckling, the limits for b/t-ratios for class C sections presented in chapter 3, may be used.

Approach 2:

Sufficient strength and ductility is supplied by the composite elements. They are so designed that they exhibit sufficient rotational capacity achieved by yielding of the steel parts and of the reinforcing steel.

By the plastic design rules laid down in chapter 4.5 a failure of the concrete parts in compression or shear proceeding the yielding of the steel parts shall be avoided

In case of mixed structures, consisting of composite frames with either steel or concrete infills, where the dissipative elements are represented by composite parts shall be so designed that they develop the sufficient over strength to localise the energy dissipation in the infills.

4.4 Behaviour factor q

The behaviour factor q introduced in Clause 4.3 of part 1.1 to account for energy dissipation capacity takes the values according to the structural types in fig. 4.3.2 provided the provision in Clause 4.5 of part 1.3 are respected.

In case of highly regular buildings to be built in regions defined by National Authorities and having earthquake resistant structures made from rolled sections or hollow sections infilled with concrete and conforming to the types listed in fig. 4.3.2 (K-bracing excepted) a behaviour q = 2,0 may be adopted without taking into account the detailing rules given in Clause 4.5 of part 1.3 and the limitations given in Clause 3.2 of part 1.3. The general safety requirements set forth in part 1.1 may then be deemed to be satisfied by the strength assessments according to Eurocode 4.

4.5 Design criteria and detailing rules for dissipative zones

4.5.1 Design criteria

For dissipative composite elements where the strength of the concrete parts is neglected (approach 1) the design criteria according to Clause 3.5.1 of part 1.3 apply. For dissipative composite elements where the concrete part is taken into account (approach 2) the following rules apply:

1 In elements subjected to bending the stiffness and yielding strength of the steel parts shall be proportioned so that yielding of the steel parts can be ensured before the concrete parts fail in compression.

2 In elements subjected to shear forces the total shear shall be resisted by the steel beam only.

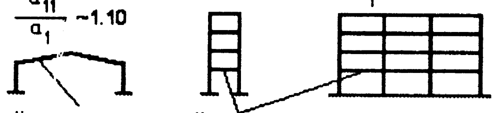
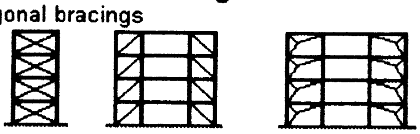
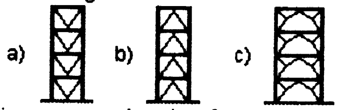
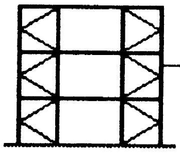
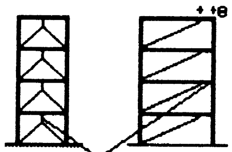
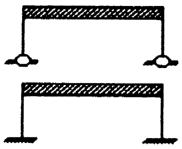
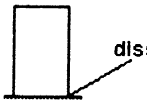
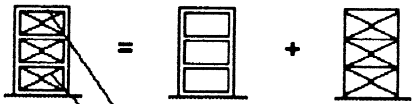
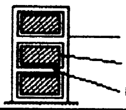
3 Elements subject to compression shall be designed such that either the concrete parts are totally encased by the steel parts or the concrete parts are confined by close spacing of the stirrups according to Clause 2.2,

4 The shear connection between concrete parts and steel parts in composite elements shall be ductile, (see EC4)

5 Any connection between dissipative concrete infills and non-dissipative composite parts that is designed for over strength shall be verified taking into account the tension stiffening of the cracked concrete parts.

6 The steel reinforcement in the concrete parts of composite elements in tension shall be sufficiently anchored according to the rules laid down in <u>Chapter 2</u>.

For dissipative reinforced concrete parts in mixed structures the design criteria laid down in chapter 2 shall be adopted.

	high regularity	medium regularity
1. Frame structures $\frac{\alpha_{11}}{\alpha_1} \sim 1.10$ $\frac{\alpha_{11}}{\alpha_1} \sim 1.20$  diss. zones diss. zones = bending zones	$q=5\ \frac{\alpha_{11}}{\alpha_1}^*$	$q=4\ \frac{\alpha_{11}}{\alpha_1}^*$
2. Concentric truss bracings Diagonal bracings  diss. zones = tension diagonals only	q=4	q=3
V - bracings  diss. zones = tension & compression diagonals	q=2	q=1.5
K - bracings  non dissipative	q=1	q=1
3. Eccentric truss bracings  $\frac{\alpha_{11}}{\alpha_1} \sim 1.10$ diss. zones = bending or shear zones	$q=5\ \frac{\alpha_{11}}{\alpha_1}^*$	$q=4\ \frac{\alpha_{11}}{\alpha_1}^*$
4. Cantilever structures  diss. zones in the columns  Restrictions. $\lambda \leq 1.5$; $\Theta \leq 0.2$ (see clause 3.5.7.1)	q=2	q=1.5
5. Cores or walls in reinforced concrete  dissipative zones	chapter 7	chapter 7
6. Dual structures  frames with diss. bending zones bracings with diss. tension zones	$q=5\ \frac{\alpha_{11}}{\alpha_1}^*$	$q=4\ \frac{\alpha_{11}}{\alpha_1}^*$
7. Mixed structures made from steel frames with reinforced concrete infills  composite frames diss. reinforced concrete or steel infills diss. joints	q=2	q=1.5

4.5.2 Detailing rules for structural elements

The detailing rules required for achieving the behaviour factors q may be taken from Clauses 3.5.4, 3.5.5, 3.5.6 3.5.7, 3.5.8, 3.5.9, of part 1.3

For steel parts the detailing rules in chapter 3, for concrete parts the detailing rules in chapter 2 are referred to.

For the shear connection in composite elements the rules in Eurocode 4 laid down for plastic design shall be applied.

4.6 Specific control measures

The relevant rules for steel and concrete structures laid down in Chapter 2 and Chapter 3 apply.

SPECIFIC RULES FOR TIMBER STRUCTURES

5.1 General criteria

Timber elements and glued joints shall be designed and verified to behave linearly without demand for hysteretic energy dissipation under seismic actions.

In structures having joints with mechanical fasteners, their plastic behaviour and capacity to dissipate energy may be taken into consideration, provided that the behaviour of the joints under cyclic loading is determined by tests either on single joints or on whole structures or parts thereof.

5.2 Materials

Eurocode No 5, chapter 3, applies.

Only mechanical fasteners and connections with appropriate low-cycle fatigue properties are allowed to be used, except for type A structures.

5.3 Structural types

Based on the ductility and the capacity to dissipate energy under seismic action, distinction shall be made between the following types:

Type A: Non dissipative structures, such as

- structures without or with only a few joints with mechanical fasteners
- arches with hinged joints
- cantilever structures with built-in columns
- structures with diaphragms solely with glued joints

Type B: Low dissipative structures, such as

- frames or beam-column structures with semi-rigid joints between all members and the foundations
- trussed frame structures with mechanical fasteners

Type C: Medium dissipative structures, such as

- structures with diaphragms resisting the horizontal forces, connected by nails. Panels may be either glued or nailed.

5.4 Behaviour factors and damping ratio

The values for the q-factors given in Table 5.4 shall be applied.

Table 5.4: Value's for the behaviour factor q

5.5 Safety verifications limitations detailing

For safety verifications the relevant provisions in chapter 4 of part 1.2 and in Eurocode No 5 apply with the following additions and modifications:

1. In diaphragms the strength of the panels shall be so high that failure will take place in the joints, not in the panels.

2. If q-values higher than 1.0 are taken into account, the members shall be designed such that their load-bearing resistance is higher than that of the connections.

3. kmod for instantaneous load apply.

4. For ultimate limit state verifications the partial safety coefficients for material properties γ_m from Eurocode No 5 table 2.3.3.2 for fundamental load combinations apply.

5. Compression members and their connections which may fail due to deformations caused by load reversals shall be so designed as to retain their original position at all times.

6. Bolts shall only be used in secondary members, and they shall be tight and tight fitting in the holes.

7. Smooth nails shall not be used.

5.6 Limitation of damage

The provisions given in part 1.2 (Clause 4.2) apply.

5.7 Control of design construction and use

The provisions given in part 1.1 apply.

SPECIFIC RULES FOR MASONRY STRUCTUES

6.1. General

1. Scope
2. Criteria for ensuring compliance with the general requirements
3. Partial coefficients for materials

6.1.1 Scope

The present chapter apply to masonry structures designed in accordance with Eurocode 6 definitions of: unreinforced masonry, confined masonry and reinforced masonry. The following rules are additional for use in seismic regions only.

Due to its low tensile strength and low ductility, unreinforced masonry is not suitable for high seismic actions. However, its association with reinforcing steel can provide ductility and limit the strength degradation under cyclic actions. The reinforcing techniques considered by Eurocode 6 and referred to in this Chapter are called "confined masonry" and "reinforced masonry" according to the definitions given in Clause 6.3.

6.1.2 Criteria for ensuring compliance with the general requirements

1 No collapse and low-susceptibility to damage

The detailing and verification rules contained in this chapter ensure the required degree of reliability against collapse.

Because of the rigidity of the masonry walls, and their concurrent structural and non structural role, the susceptibility to damage is limited by the same rules, with the addition of only few further measures as specified in Clause 6.6

2 Specific aseismic measures

In order to minimise the uncertainties inherent in aseismic design, and their consequences, the following conditions shall be met:

i) The behaviour of the structural system as a whole shall be ensured by effective horizontal connections between the walls and the floors.

ii) Bearing walls shall be provided with stiff foundations, so that they can be considered as built-in at their base.

iii) The design documents shall point out the detailing whose effectiveness is essential for seismic resistance.

6.1.3 Partial coefficients for materials

Partial coefficients for masonry strength are the following:

The partial coefficient for steel is: $\gamma s=1.0$.

6.2 Materials

1 Artificial units

The rules presented in this chapter apply to masonry made with solid, perforated, hollow, cellular and horizontally perforated units according to Eurocode 6.

Perforated, hollow, cellular and horizontally perforated units should not have more than 50 % by volume of holes.

The minimum thickness of their face shells and cross webs shall not be less than .15 mm. Vertical webs in hollow and cellular units shall extend to the entire length of the unit. Units different from the above can be used in reinforced masonry systems provided they comply with the tests foreseen in Clause 6.3.4.

2 Natural stone units

Stone masonry is allowed in seismic regions, provided units are square dressed with parallel horizontal faces.

3 Minimum strength of units

Both artificial and natural units shall have a mean compressive strength not less than the following values obtained from tests carried out in accordance with the relevant provisions of Eurocode 6.

With reference to the normal bedding position in the masonry:

a) in the vertical direction: $fb = 2{,}5\ N/mm^2$

b) in the horizontal direction in the plane of the wall: $fb = 2.0\ N/mm^2$

4 Mortar - Grout - Concrete infill

For unreinforced and confined masonry, only mortar types M20 - M15 - M10 - M5 - according to Eurocode 6) are allowed in seismic regions. National Authorities can allow, depending on the seismicity of the region and the importance of the building, the use of mortar type M2.

For reinforced masonry, only mortar types M20 - M15 - M10 are allowed.

Grout and concrete infill shall comply with the relevant provisions of Eurocode 6.

5 Reinforcing Steel

Only steel type A as defined in Eurocode 2 shall be used.

6 Bonding patterns

In addition to the rules given in Eurocode 6, for the application of the present chapter concerning all types of masonry made by artificial or natural units the following specifications shall be met.

The courses shall be horizontal.

- Vertical joints between adjacent courses shall be staggered not less than one third of the length of the unit (one third running bond), and the joints shall be filled with mortar.

- joints between orthogonal walls shall be fully bonded.

- When the thickness of the wall requires more than one unit, a staggered pattern shall be used in order to realise not less than one third running bond across the wall.

6.3 Types of structural masonry for buildings

Unreinforced, confined, and reinforced masonry built in accordance to EC6 are allowed in seismic regions, with the following additional limitations:

1) Unreinforced masonry
2) Confined masonry
3) Reinforced masonry
4) Reinforced masonry systems

6.3.1. Unreinforced masonry

The thickness of seismic resisting walls shall not be less than

for artificial units walls 300 mm,(suggested)

for natural units walls 400 mm,(suggested)

The ratio between the storey height and the wall thickness shall not be greater than the following values:

for artificial units walls 12,(suggested)

for natural units walls 9,(suggested)

Horizontal steel-ties shall be provided in the plane of the wall, at every floor level and in any case not spaced more than 4 m.(suggested

This type of structural masonry is not allowed in buildings having more than 2 storeys in seismic regions where the peak ground acceleration is greater than ag = 0,25 g, unless the conditions for the simple building are fulfilled (Clause 6.4.2.1).

6.3.2 Confined masonry

This type of masonry shall be confined by horizontal and vertical structural reinforced concrete tie-beams adequately bonded together and anchored to the elements of the main structural system.

In order to obtain an effective bond between tie-beams and masonry, casting of the concrete shall follow the construction of the masonry wall.

The cross section of both horizontal and vertical tie-beams shall be not less than. 150x150 mm.(suggested)

Vertical confining elements shall be placed at both sides of any wall opening with an area of more than __ m2, at every intersection between walls, and within the wall if necessary in order not to exceed the spacing of: 5 m.(suggested)

Horizontal tie-beams shall be placed at every floor level and be connected to the floor slabs; in any case their vertical spacing shall not exceed: 5 m.(suggested)

In each vertical or horizontal tie-beam the reinforcement shall be not less than 2.5 cm (suggested) of cross section. If more than one steel bar is adopted in the tie-beams, the reinforcement shall be put together by regularly spaced stirrups.

Continuity of the reinforcement shall be achieved by 60 diameters overlap.

Wall thickness shall be not less than: 240 mm.(suggested)

The ratio between the storey height and the wall thickness shall not be greater than: 15.

6.3.3 Reinforced masonry

Reinforced masonry is characterised by the presence of reinforcement diffused within the masonry according to the following rules.

Horizontal reinforcement shall be placed in the bed joints or in suitable grooves in the units, and spaced not more than 600 mm.(suggested)

Special units shall provide place for the reinforcement needed in the lintels and in the parapets.

Steel bars with diameter not less than 4 mm, bended around the vertical bars at the edges of the wall shall be used.

The minimum percentage of horizontal reinforcement spread in the wall, referred to the cross section of the wall, shall not be less than 0.5l1000.(suggested)

Vertical reinforcement shall be located in appropriate pockets, cavities or holes of the units.

Vertical reinforcement with section not less than, 4 cm2 shall be arranged at both free edges of every wall element, at ever wall intersection, and within the wall, if necessary, in order not to exceed the spacing of 4m.(suggested)

The minimum percentage of vertical reinforcement spread in the wall, referred to the cross section of the wall, shall not be less than: 0.5/1000.(suggested)

The parapets and lintels shall be regularly bonded to the masonry of the adjoining walls and linked to them by horizontal reinforcement.

Wall thickness shall not be less than: 240 mm.(suggested)

The ratio between the storey height and the wall thickness shall not be greater than:

*) The provisions contained in this Clause will be reviewed when the relevant part of Eurocode 6 becomes available.

6.3.4 Reinforced masonry systems.

Reinforced masonry systems may be industrially produced, with a set of standard and special units supplied with pockets and grooves appropriate to accommodate the reinforcement, and a corresponding consistent set of properly studied details.

In this case the system may violate some of the specifications of Clause 6.3.3 provided it is submitted to appropriate experimental tests, according to the indications of agreed Standards.

The complete qualification documentation of the system shall be officially approved and shall contain all the relevant informations for design and construction of the system.

In this document the shape of all the unit types, the assembling procedures, the arrangement of the reinforcement, the mechanical characteristics, etc. shall be described.

6.4 Constructional rules

The constructional rules can be classified in:

1) General rules
2) Particular rules for "Simple Buildings"

6.4.1 General rules

Buildings constructed using masonry shall behave as three-dimensional structures made up of floors and walls with all the elements contributing to resist the applied load.

Shear walls shall be provided in 2 orthogonal directions. The floors shall be verified to be able to transfer horizontal actions to the walls.

The following rules apply to floors:

Any type of floors can be adopted, provided the general requirement of continuity is satisfied.

In particular for reinforced concrete slabs a layer of continuous concrete having a thickness of not less than 40 mm (suggested) shall be provided, unless other means assure effective diaphragm action.

Floors shall be reinforced in 2 orthogonal directions and reinforcement shall be anchored in the perimeter tie-beams confining the floor.

In case of parallel ribbed floors, intermediate ties shall be placed orthogonally to the main reinforcement, so as not to exceed the spacing of 5 m (suggested) between ties. These shall contain not less than 6 cm2 (suggested) of reinforcement, effectively anchored at both ends.

Precast slabs shall be reinforced in 2 orthogonal directions and connected with the supporting beams and each other with shear connectors, so that the whole forms a continuous diaphragm.

Steel and timber floors may be used if they fulfil the requirements of effective diaphragm action in transferring the horizontal actions to the shear walls.

see also: Clause 6.4.2 of part 1.3.

6.4.2 Particular rules for "Simple Buildings"

For the purpose of the present Code, if a building satisfies, in addition to the prescriptions of Clause 6.4.1 of part 1.3 the ones contained in the following, it is defined a "Simple Building".

The number of storeys above ground does not exceed the following values:

The plan is approximately rectangular, with sides ratio not less than 0.25 (suggested) and with projections or recesses from the rectangular shape not greater than 15 % (suggested) of the side parallel to the direction of the projection, and less than 2 m.(suggested)

At least 75 % of the vertical load is supported by walls.

The in-plan-layout of the more important walls is symmetrical with respect to two orthogonal axes.

Two parallel walls with not less than 50 % of the length of the building in the direction of those walls shall be located at a distance from each other greater than 75 % of the other plan dimension of the building.

The difference in mass and in wall horizontal cross section between the storeys does not exceed 20 %.

Tie-beams and minimum reinforcement are provided according to the rules given for the corresponding structural type (unreinforced, confined, reinforced masonry).

The horizontal cross section of the lateral load resisting walls, in each of the orthogonal directions, given in percent of the total floor area above the level considered, at every floor is not less than the values of the following table:

The wall elements with height-length ratio greater than the value given in Clause 6.6.1 of part 1.3 shall not be taken into account in the evaluation of the lateral load resisting area.

For unreinforced masonry buildings, the walls in one direction shall be connected with the walls in the orthogonal direction at a maximum distance of 7 m.(suggested)

6.5 Behaviour factor damping ratio and fundamental period

The following values of the behaviour factor q shall be adopted:

Unreinforced masonry	1.5	(suggested)
Confined masonry	2.0	(suggested)
Reinforced masonry	2.5	(suggested)

Values of q for reinforced masonry systems shall be derived from the results of the ductility tests referred to in Clause 6.3.4 of part 1.3 and shall be officially approved.

With reference to Clause 4.3.1, of part 1.1 for masonry buildings the damping factor ζ = 8 % can be adopted.

In the absence of an analytical evaluation of the fundamental period of the building, for the purpose of application of the linear analysis design spectra, T shall be taken equal to T2 (Clause 4.2.1.2 of part 1.1).

6.6 Safety verification

1. Definition of seismic-resistant elements
2. Methods of analysis
3. Safety verification

6.6.1 Definition of seismic-resistant elements

In a masonry building the following elements can act as seismic load-resisting elements.

a) Masonry walls, provided the height/length ratio is not greater than the value given in the table below.

If the height/length ratio is greater than the above values, the element shall not be taken into account as a resisting element.

b) Horizontal connection elements, acting as coupling beams between two vertical elements. The masonry parapets, if regularly bonded to the adjoining walls, connected to the floor tie beam and to the lintel below, may be considered as coupling beams.

6.6.2 Methods of analysis

A static analysis procedure (i.e. the simplified dynamic analysis in Clause 5.3.1.2 of part 1.1) is considered appropriate to masonry buildings. A linear analysis can generally be used modelling the whole building and considering the floors rigid in their plane. Flexural, shear and axial deformability taken into account in evaluating the stiffness of the elements.

If the structural model takes into account the coupling beams, a frame analysis can be used for the determination of the action effects in the vertical and horizontal structural elements.

More refined methods (e.g. finite elements) may also be used.

The distribution of the total base shear among the walls, as obtained by the linear analysis, may be modified, provided that global equilibrium is assured and the action in any wall is neither reduced more than 30 % (suggested) nor increased more than 50 %.(suggested)

6.6.3 Safety verification

For the safety verification against collapse described in Clause 4.1.1.1 of part 1.2 the design resistance of every seismic resisting element shall be evaluated according to chapter 4 of Eurocode 6 and considering the values of presented in Clause 6.1.3.of part 1.3

In the case of the "simple buildings" fulfilling the particular requirements given in Clause 6.4.2 of part 1.3 the above verification is implicit in the design procedure.

6.7 Limitation of damage

The provisions given in Clause 4.2 of part 1.2apply. For "simple buildings" these conditions are considered to be implicitly satisfied.

6.8 Control of design construction and use

Provisions contained in EC 6 Chapter 6 shall be adopted.

SPECIFIC RULES FOR MIXED STRUCTURES

7.1 General

1 Scope and definition

The following provisions apply for structural systems designed to resist horizontal seismic forces, whose parts (frames, diaphragms, infills) and/or elements (columns, slabs, walls) consist of different materials, but provide a single homogeneous resisting system in a vertical plane (mixed structures).

The provisions don't apply for "dual structures", whose parts or elements may also consist of different materials, but are able to provide more than one resisting system in a vertical plane, as for example the association of wall and frame elements.

Mixed wall structures consisting of masonry walls confined by reinforced concrete or steel ties (confined masonry) are dealt with in Chapter 6.

2 General requirements

The different elements shall be assembled in an appropriate way to form a resisting structural system.

7.2. Materials

The different materials of mixed structures shall meet the requirements given in the relevant Eurocodes and in Chapters 2, 3, 4, 5, and 6.

7.3 Structural types

The provisions of this Chapter apply for the following mixed structures:

TYPE A:

Mixed structures consisting of reinforced concrete or steel frames filled by normal or lightweight reinforced concrete or brick or block masonry walls

TYPE B:

Mixed structures consisting of reinforced concrete cores or walls and steel columns connected with reinforced concrete slabs

TYPE C:

Mixed structures consisting of triangulated timber frames filled by masonry.

7.4 Behaviour factors

TYPE A - structures:

The behaviour factors q given in Chapters 2 and 3 of this Code for bare reinforced concrete and steel frames respectively shall, when needed, be reduced to account for the influence of infill walls.

Reduced q-factors shall be considered if

(a) non-uniform distribution of infill walls in elevation (vertical irregularity) causes increased ductility demand in frame elements of the "less-infilled" floors or bays,

(b) additional shear forces locally acting on columns (near column/beam joints) due to the frame/infill interaction may cause a reduction of the shear ratio of columns and, therefore, may also reduce their ductility.

TYPE B - structures:

(a) If, according to a linear analysis, more than 60 % of the base shear force is carried by the reinforced concrete core(s), then the core(s) shall be designed so as to be able to carry the total shear force; the appropriate q-factors given in Clause 2.1.4 of Chapter 2 apply.

(b) If, according to the structural analysis, less than 60 % (suggested) of the total seismic force is carried by the concrete core(s), then the q-factors given in Clause 3.4 of Chapter 3 apply.

TYPE C - structures

The behaviour factors given in Chapter 5 of this Code apply.

7.5 Safety verifications limitations detailing

TYPE A - structures

Safety verifications of reinforced concrete and steel frame elements and for infill walls concerning out-of-plane bending and connections to the frames shall be carried out according to Chapter 2, 3 and 6 respectively.

Additional shear forces and bending moments acting on columns, due to frame/infill interaction, shall be taken into account when dimensioning columns.

No safety verifications concerning in-plane bending and shear of infilling walls are needed, unless these walls are taken into account for strength verifications of the entire structural system. In this case the relevant provisions for walls given in Chapter 2 and 6 apply.

Limitations and detailing rules given in Chapter 2, 3 and Chapter 6 apply.

TYPE B - structures

Safety verifications of reinforced concrete walls and steel frame elements shall be carried out according to Chapter 3

It shall be verified that the concrete diaphragms are able to distribute the horizontal forces among the resisting vertical elements.

In the case the total shear force is considered to be carried by the reinforced concrete cores, the columns shall be designed based on the results of a static analysis according to the rules contained in Chapter 3.

Limitations and detailing rules given in Chapters 2 and 3 apply.

TYPE C - structures

Safety verifications of timber elements and of infill walls concerning out-of-plane bending and connections to the frames shall be carried out according to Chapter 5 and 6.

No safety Verifications concerning infills are needed.

Limitations and detailing rules given in Chapter 5 apply.

7.6 Limitation of damage

TYPE A - structures

The inter storey drift of reinforced concrete and steel infilled buildings may be calculated according to Clause 5.5 of part 1.1 and shall be limited, according to Clause 4.2.2 of part 1.2, considering the infills as a brittle material attached to the structure.

TYPE B - structures

The general rules contained in Clause 4.2 of part 1.2 aiming at limitation of damage apply.

TYPE C - structures

In addition to the limitations contained in Clause 4.2.2 of part 1.2 light wire meshes should be placed on both sides of timber-masonry infilled walls (appropriately anchored into the walls) and be protected appropriately.

7.7 Control of design, construction and use

The respective sections of Chapters 2, 3, 5 and 6 and of part 1.1 of this Code apply.

SPECIFIC RULES FOR STUCTURAL ELEMENTS

8.1 General

1 Scope and definitions

This chapter is concerned with specific rules for non-structural elements in buildings. Non-structural elements are architectural, mechanical or electrical elements, systems and components that, whether due to lack of strength or to the way they are connected to the structure, are not in a position to affect the stiffness of this latter; consequently - except for their weight -they are not included in the analysis of the structure.

2 General requirements

Architectural, mechanical and electrical elements, systems and components and their connections shall be designed and constructed to resist seismic action

without endangering persons by failing

without affecting the main structure

without affecting other non-structural elements

remaining operational when being part of vital facilities.

Those "non-structural" elements which cannot be clearly restrained from interacting with structural elements shall be introduced in the model used for the analysis.

8.2 Types of non-structural elements

Non-structural elements may be classified into the following types

8.2.1 Architectural components

Architectural, components are

1) Appendages, such as

exterior non-bearing walls

wall attachments

veneers

roofing units

free standing components, such as chimneys, antennae a.o.

2) Partitions such as

stairs and shafts

vertical shafts

horizontal exits

public corridors

non-bearing walls

structural fireproofing

suspended ceilings

3) Fixtures. such as

built-in cupboards

shelves

8.2.2 Mechanical/electrical components

Mechanical/electrical components are

1) Heating/cooling components such as

boilers, furnaces a.o.

chimneys, flues, vents a.o.

communication systems

tanks

ducts and piping distribution systems

2) Electrical components such as

electrical ducts and cable systems

electrical supervision or safety systems

lighting fixtures

8.2.3 Storage elements or storage material

8.3 Safety verification limitations detailing

8.3.1 General

Safety verifications shall be carried out for elements/components and their connections that might, in the case of failure, cause risk to persons.

Specific provisions, dependent on the seismicity level of the zone, will be given by the competent National Authorities.

8.3.2 Safety verifications limitation detailing for elements/components

1) Individual elements/components as described in Clause 8.2.1-8.2.3 and their connections shall be verified according to the provisions given in Clause 5.4 of part 1.1, in Clause 4.1 of part 1.2 and the other Eurocodes to have adequate strength and overall stability.

2) The horizontal seismic action on any element/component shall be applied at its centre of gravity and be assumed to act in any horizontal direction. Where required, the vertical seismic action (Clause 4.2.1, part 1.1) shall also be applied.

The horizontal and vertical seismic actions shall be calculated according to Clause 5.4 of part 1.1.

3) For panels, their strength and the strength of their connections under seismic actions shall be verified both in their plane as well as orthogonally to it.

4) Elements and components shall be connected together and attached to the structure so as to preclude that failure of a single element may automatically involve a generalised failure of the interconnected items.

5) Connections and attachments

   a) Non-structural elements/components and their connections shall be arranged such as not to affect the deformations of the main structure.

   b) Connections shall be checked to adjust for the design deformations of the structure (Clause 4.1 of part 1.2), either elastically or with an adequate ductility reserve.

   c) For small sized overlapping elements, such as tiles, it is sufficient to attach only the individual elements of a group, necessary to ensure the attachment of the whole group.

   d) In the absence of exact verifications hanging or swinging - type fixtures shall have a safety cable attached to the structure at each support point, capable of supporting 4 times their weight.

8.3.3 Storage elements or storage material

Storage elements or storage material shall be arranged in a self-restraining way or be provided with adequate restraints If i.e cables or other attachments to the structural in order not to loose their overall stability (sliding, overturning, uplifting), causing

danger to the stability of the main structure or of non-structural elements or

blocking of emergency areas.

8.4 Limitation of damage

The provisions given in Clause 2.1.2 and 2.2.2 of part 1.1 and Clause 4.2 of part 1.2 apply.

8.5 Control of design construction and use

The provisions given in part 1.1 apply.