SESSION E

Fire and Steel Buildings
Earthquake and Steel Buildings

Feu et bâtiments en acier
Séismes et bâtiments en acier

Feuer und Stahlbauten
Erdbeben und Stahlkonstruktionen

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Sécurité au feu: résultats de la conférence de Luxembourg, avril 1984

Brandsicherheit: Lehren aus der Konferenz in Luxemburg, April 1984

Fire Safety: Main Lessons from the Conference of Luxemburg, April 1984

Paul BORCHGRAEVE
Directeur
CBLIA
Bruxelles, Belgique

Paul Borchgraeve, né en 1931, est ingénieur commercial de l'Université Catholique de Louvain. Carrière en promotion et développement des marchés en sidérurgie. Depuis 1972, Directeur du Centre belgo-luxembourgeois d'Information de l'Acier. Président du Comité Promotion Acier d'Eurofer. Il anima le Comité Programme de la Conférence 1984 «Sécurité au feu des constructions acier».

RÉSUMÉ

Au cours des quinze dernières années, on a assisté à l'indispensable approfondissement et à l'amélioration des connaissances du comportement au feu des structures acier. Aujourd'hui, une nouvelle étape plus décisive est franchie dans le progrès. Les méthodes rationnelles d'évaluation du risque d'incendie et la modélisation d'actions thermiques et de réponses des structures en acier ouvrent des perspectives nouvelles de solutions plus compétitives et offrant une sécurité donnée et uniforme.

ZUSAMMENFASSUNG

Im Laufe der letzten fünfzehn Jahre konnte die unumgängliche Vertiefung und Erweiterung der Kenntnisse des Brandverhaltens von Stahlkonstruktionen beobachtet werden. Heute wird eine noch entscheidendere Entwicklungsstufe durch den Fortschritt überbrückt. Die rationellen Verfahren zur Bestimmung des Brandrisikos und zur Modellierung der Wärmewirkung und des Tragverhaltens von Stahlkonstruktionen eröffnen neue Perspektiven für wettbewerbsfähigere Lösungen und bieten eine festgelegte und einheitliche Sicherheit.

SUMMARY

During the last fifteen years we have witnessed a necessary phase of thorough study to deepen our knowledge of the phenomena related to the fire safety of steel structures. Today, a new step forward – to be more decisive – has been taken. The establishment of rational methods of fire risk assessment and the simulation of thermal effects and fire behaviour of steel structures are opening new prospects for more competitive solutions offering specific and uniform safety.

1. ORGANISATION ET OBJECTIFS DE LA CONFERENCE.

Une Conférence Internationale s'est tenue en avril 1984 à Luxembourg sur le thème "Sécurité au feu des constructions en acier : conception pratique".

L'initiative a été possible par la conjonction des efforts de plusieurs parties engagées à des titres divers dans des actions relatives à la sécurité au feu des constructions acier :

- La Commission des Communautés Européennes dans le cadre de ses activité de recherche CECA-Acier. Les programmes dans le domaine du feu (recherches, essais, enquêtes, constructions de stations d'essais au feu, travaux d'élaboration de méthodes de calcul) auxquels la CECA a apporté une importante contribution, représentent depuis 1966 un investissement total de 3 millions d'écus.
- La Convention Européenne de la Construction Métallique (CECM) et plus particulièrement son Comité Technique 3, est à la base de l'élaboration des "Recommandations européennes pour la sécurité au feu des structures en acier".
- Les Centres d'Information et de Promotion de l'Acier des pays de la Communauté. Organes de la Sidérurgie de leurs pays mandatés pour l'information et la promotion en matière d'utilisation des produits sidérurgiques, les Centres ont à leur actif de nombreuses publications et participent sous diverses formes à la diffusion des connaissances dans le domaine de la sécurité au feu des constructions en acier.

Les objectifs de la Conférence se voulaient dynamiques et pratiques :

1° montrer que le degré des connaissances dans le comportement au feu des structures acier s'est fortement amélioré et qu'il est possible d'offrir des niveaux de sécurité tout à fait comparables à ceux atteints avec des constructions plus traditionnelles;

2° montrer les progrès considérables accomplis ces dernières années dans l'approche rationnelle ou analytique (rational approach or fire engineering) par le biais de méthodes et de modèles de calcul et présenter les possibilités d'évolution vers des méthodes d'analyse du risque beaucoup plus proches de la réalité;

3° sensibiliser les décideurs privés et publics, les Autorités de réglementation, les Sapeurs-Pompiers et les Assureurs à cette approche pour l'évaluation pratique de la performance des structures acier et des structures mixtes acier-béton à la sollicitation incendie.

2. ENSEIGNEMENTS PRINCIPAUX.

2.1. La conception de la sécurité basée sur l'approche rationnelle ou analytique gagne du terrain

a) Les informations nécessaires pour concevoir rationnellement une structure du point de vue de la sécurité à l'incendie ne peuvent pas être fournies par les seuls résultats d'essais normalisés sur lesquels sont encore fondés les codes et les réglementations.

En effet, le comportement des structures en cas d'incendie est un problème extrêmement complexe. Les essais ne suffisent pas pour représenter le comportement de tous les éléments de structure dans des conditions d'incendie, car l'évaluation de la réaction de la structure doit également rendre compte des diverses contraintes dues au système de construction. De plus, la nécessité d' effectuer un essai pour tout nouvel élément de structure n'est pas une procédure très rationnelle, étant donné que ces essais d'incendie sont longs et coûteux.

b) On a insisté pour que soient abandonnés le système de classification actuel (durée exigée selon la courbe ISO et spécifiée dans les règlements en multiples de 30 minutes) et l'essai normalisé de résistance au feu (toujours selon ISO 834) qui représentent l'un et l'autre de sérieuses lacunes.

c) Les prévisions analytiques des réponses thermiques et structurales et des risques d'incendie deviennent de plus en plus nécessaires. L'évaluation pratique de la résistance au feu des structures acier peut désormais s'opérer par de nouvelles méthodes de calcul. Le recours à l'ordinateur rend cette approche commode et pratique.

d) Plusieurs méthodes sont disponibles :

- Recommandations européennes élaborées par la Convention Européenne de la Construction métallique.

En raison des limites de la plupart des normes ou règlements nationaux concernant la sécurité au feu, ces Recommandations se sont volontairement limitées à présenter un modèle de calcul qui n'a pour objet que de dégager des résultats identiques à ceux qui seraient obtenus par des éléments de structures testés dans un four d'essais. Ce modèle pourra être facilement adapté à un calcul du comportement de la structure acier dans son ensemble lorsque l'incendie normalisé sera abandonné.

Le calcul de la durée de stabilité au feu d'un élément de structure acier est divisé en deux parties indépendantes :
. calcul de la température atteinte après une certaine durée d'exposition à l'incendie normalisé;
. calcul de la température critique ou de ruine, c.à.d. à laquelle cet élément s'effondrera.

- Dimensionnement pratique de colonnes mixtes acier-béton

Programmes de dimensionnement par éléments finis tridimensionnels et non linéaires pour l'étude des problèmes thermiques et d'instabilité des colonnes mixtes de quatre types :
. profils ouverts I enrobés complètement de béton;
. profils ouverts I avec enrobage de béton entre les ailes;
. profils creux remplis de béton;
. profils composés d'un noyau central en acier massif enrobé de béton et contenu dans un tube mince en acier.

- Méthode CECM-ECCS de calcul et de conception de planchers mixtes acier-béton avec tôle d'acier galvanisée nervurée

Méthode analytique de la capacité portante pour des exigences de durée de résistance au feu de plus de 30 minutes. Calcul de l'épaisseur minimum de la dalle et des armatures de renforcement

- Calcul de planchers mixtes acier-béton et de poutres collaborants.

Détermination de la résistance au feu et de l'épaisseur d'isolation.

- Modèle informatique AISI "Fires-T3" (FIre REsponse of Structures-Thermal-3 Dimensional Version)

Programme qui utilise la méthode des éléments finis tridimensionnels pour la prévision du transfert et de la distribution de chaleur dans les éléments en acier ou en béton armé.

- Modèle informatique AISI "Fasbus II" (Fire Analysis of Steel BUilding Systems)

Egalement basé sur la méthode des éléments finis, il est conçu pour analyser et prédire la résistance au feu de systèmes de planchers composés de poutrelles et de solives en acier et d'une dalle de béton.

- Modèle numérique ARBED

Simulation et prédiction de la résistance au feu des constructions en acier et des constructions mixtes, pour n'importe quelle combinaison d'acier et de béton. Utilise la théorie des éléments finis.

- Approche suédoise dite des incendies réels

Modèle analytique des structures et parois portantes exposées au feu, par représentation directe axée sur les caractéristiques thermiques de l'incendie de compartiment ayant atteint son intensité maximale.

- Conception de la sécurité basée sur des calculs de probabilité

Cette conception constitue la base du code modèle en préparation au Conseil International du Bâtiment et a été appliquée à une norme d'évaluation des bâtiments industriels en Allemagne Fédérale (Norme DIN 18230).

Les concepteurs, les organismes de prévention et les assureurs disposent donc aujourd'hui de nouveaux moyens d'approche dynamiques, rapides et fiables et fournissant des solutions assez simples à des problèmes très complexes.

2.2. Conséquences financières des incendies

Des enquêtes très sérieuses ont été réalisées dans les pays nordiques sur les conséquences financières d'incendies réels. Des enquêtes semblables sont actuellement en voie d'achèvement en France et aux Pays-Bas.

Les résultats révèlent que la propagation de l'incendie et les pertes dues au feu dépendent d'autres facteurs que la seule résistance au feu de la structure portante. Une multitude de paramètres différents, souvent en corrélation les uns avec les autres, exercent une influence. Ce sont par exemple le type d'activité, le type de bâtiment, la superficie du bâtiment, les mesures de sécurité actives, etc. Il a pu être démontré qu'il n'y a pas de différences quant aux dégâts dus au feu entre les bâtiments industriels à structure acier et les bâtiments industriels à structure béton.

L'analyse des incendies industriels démontre également que dans tous les cas où la charge au feu était élevée les dégâts ont été totaux, quel que soit le type de bâtiment ou de structure.

Enfin, à partir du résultat des enquêtes, l'analyse coût-bénéfice pour différentes mesures de protection incendie dans des bâtiments industriels conduit à l'estimation de leur rentabilité.

Il en résulte qu'on ne peut pas prétendre qu'une augmentation de la résistance au feu de la structure portante (en acier) réduira considérablement les dégâts d'incendie. Le bénéfice escompté est très inférieur au coût. Le contraire peut par contre s'avérer vrai avec des sprinklers et des cloisons.

Sur la base des connaissances qui précèdent, les investisseurs peuvent orienter plus judicieusement leur choix vers les mesures qui donnent la plus forte réduction des dégâts prévisibles en regard de leur coût.

Pour leur part, les assureurs se trouvent mieux armés pour déterminer les risques en fonction de critères plus objectifs et établir une tarification non discriminatoire selon les niveaux de risques.

3. CONCLUSIONS

Par rapport à la conception classique de résistance au feu basée sur une classification et sur des résultats d'essais normalisés de résistance au feu, l'approche rationnelle ou analytique offre une démarche logique plus rapide et plus simple pour prévoir le comportement au feu des structures en acier et des structures mixtes.

Deux avantages majeurs en découlent :

1°) des niveaux de sécurité plus réalistes;
2°) une meilleure économie générale.

Pour tirer le meilleur parti des perspectives mises en lumière par la Conférence de Luxembourg d'avril 1984, l'accent doit être mis dans le futur sur les trois objectifs suivants :

- un effort d'information pour faire connaître les progrès obtenus dans la sécurité au feu des constructions en acier;

- une action visant à adapter les règlements et codes de construction aux progrès acquis et à encourager l'approche rationnelle ou analytique par l'application de méthodes et de modèles de calcul;

- l'orientation des programmes de recherche vers l'étude des exigences fonctionnelles basées sur des objectifs précis de sécurité au feu.

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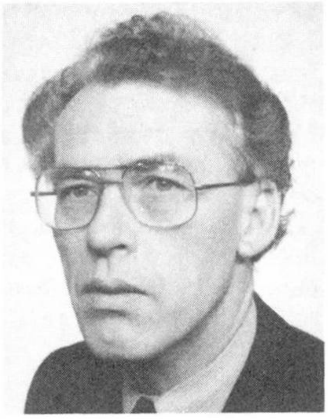
Design Methods for Fire exposed Steel Structures

Méthodes de calcul au feu des constructions métalliques

Methoden für die rechnerische Beurteilung des Brandverhaltens von Bauteilen

Jelle WITTEVEEN
Director
Institute TNO
Rijswijk, the Netherlands



Jelle Witteveen, born in 1933, received his civil engineering degree at Delft University of Technology. His research activities include studies of plastic behaviour of steel structures, policies for fire safety and fire engineering design.

SUMMARY

During the last decade much progress has been made in the analytical modelling of fire exposure and in the development of probabilistic methods of fire risk assessment. Analytical methods have been developed for the determination of the load bearing capacity of elements and structures at elevated temperatures as an alternative to the standard fire resistance test. This paper reviews design methods for structural fire safety, in particular for steel structures, which were developed in the last decade and are now being used and implemented in building regulations and structural codes.

RÉSUMÉ

Dans la dernière décade, un grand progrès a été fait dans la modélisation de l'exposition au feu et le développement de méthodes probabilistes pour l'évaluation du risque d'incendie. Des méthodes analytiques ont été développées pour la détermination de la capacité portante aux températures élevées d'éléments et de structures comme alternatives au test de résistance au feu standard. La présente contribution passe en revue les méthodes de calcul de la sécurité structurale à l'incendie, en particulier pour les structures métalliques; ces méthodes, développées durant la dernière décennie permettent de traiter des applications pratiques et d'améliorer les prescriptions en matière de bâtiments et les codes de calcul des structures.

ZUSAMMENFASSUNG

Im letzten Jahrzehnt erfolgten wichtige Fortschritte sowohl beim rechnerischen Erfassen des Brandverhaltens als auch bei der Beurteilung des Brandrisikos. Rechnerische Verfahren für die Ermittlung der Tragwiderstände von Bauteilen und von Tragwerken unter Brandeinwirkung wurden als Alternative zum Normbrandversuch (Ofentest) aufgestellt. Dieser Beitrag liefert eine Übersicht über Methoden zur rechnerischen Bestimmung des Brandwiderstandes von Tragkonstruktionen, insbesondere von Stahlkonstruktionen. Diese Methoden haben inzwischen sowohl in die Praxis als auch in nationalen Richtlinien Eingang gefunden.

1. INTRODUCTION

Fires affect the structural performance of buildings, because they change the physical and mechanical properties of materials of construction. As a consequence a fire engineering design system needs to quantify the fire exposure on the one hand and the effects of that exposure on structural behaviour on the other hand. Presently, the design system is generally based on grading of elements of construction in a standard fire resistance test. In the building regulations structural performance is defined as the minimum time for which each element would survive if it was subjected to a standard fire test. Although this grading system with the associated test procedures has been in existence for more than half a century, serious weaknesses can be observed. This applies to the rather arbitrary quantification of fire exposure, including safety considerations, as well as to deficiencies in test procedures, such as inadequate repeatability, reproducibility and simplifications with respect to actual conditions in the structure. The deficiencies in the present design system have certainly stimulated the development of rational methods of risk assessment and analytical modelling of thermal actions and structural response. This paper reviews design methods which have been developed in the last decade and are now becoming operational for practical application and implementation in building regulations and structural codes.

2. CONCEPTS IN STRUCTURAL FIRE ENGINEERING DESIGN

As discussed in the introduction, a structural fire engineering design includes two components i.e. quantification of the fire exposure (heat exposure model) and the effect of that exposure on the structure (structural response model) [1, 2]. Both models can briefly be described as follows:

a. Heat exposure model H, for the determination of the rise of temperature as a function of time. The heat exposure model is supplemented in a probabilistic way, by factors which take into account the probability of occurrence of a large fire, reliability of sprinklers, occupancy, height and volume of the building and the consequence of failure for the overall stability of the building.

b. Structural response model S, for the determination of the heat transfer to and within the structure and the ultimate load bearing capacity of the structure. The structural model may be experimental or analytical.

The design implies a proof that the structure or the structural element under a defined load and subjected to the specified heat exposure, fulfils certain functional requirements, expressed by relevant limit states. The available heat exposure models (H) (see vertical column in Fig. 1) and the structural response models (S) (see horizontal row in Fig. 1) can be characterized with respect to the type of thermal exposure and the type of structural system. The models are listed in a sequence of improved schematization, and consequently also with increased complexity of application (see chapters 3 and 4 for further details).

2.1. Heat exposure models

(H_1) A rise of temperature as a function of time according to the standard temperature time curve.
The duration of the temperature rise is equal to the "required time of fire duration", expressed in building regulations and codes.

(H_2) A rise of temperature as a function of time according to the standard temperature time curve.
The duration of the temperature rise is equal to the "equivalent time of fire exposure", a quantity which relates a non-standard or natural fire exposure to the standard temperature-time curve.

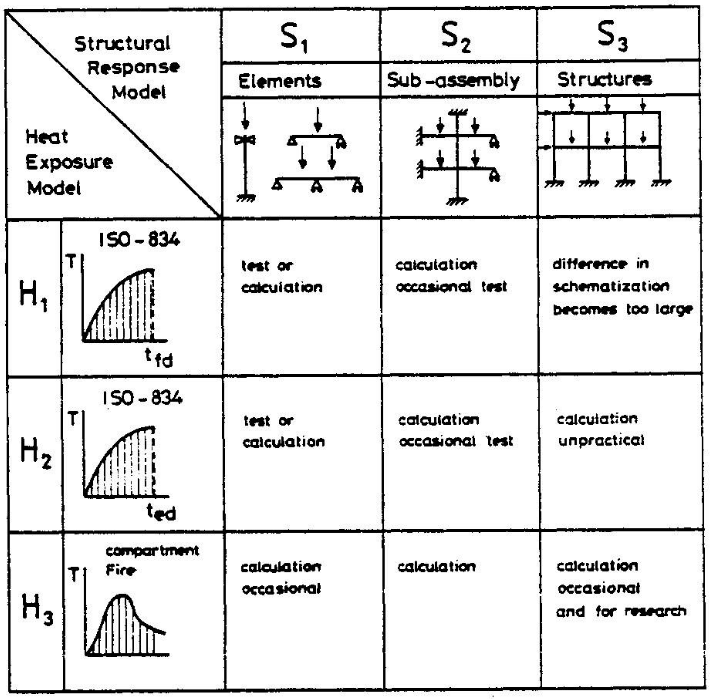
(H_3) A rise of temperature as a function of time characterized by an analytical determination of the gas temperature-time curve of a fully developed compartment fire (natural fire).

2.2. Structural response models

(S_1) The load bearing structure is idealized as a series of single members with simplified restraint conditions such as beams and columns. The model can be either experimental (standard fire resistance test) or analytical.

(S_2) The load bearing structure is idealized as a number of sub-assemblies, such as beam-column systems. Although the model can occasionally be experimental (standard fire resistance test), an analytical approach will be prevalent.

(S_3) The load bearing structure, such as a building frame or a floor slab system is analysed as a whole. The model is only suitable for analytical design.



t_{fd} = required time of fire duration
t_{ed} = design equivalent of fire exposure

Fig. 1 Matrix of heat exposure and structural response models in sequence of improved idealization [1]

Each combination of heat exposure model and structural response model, as an element of the matrix in Fig. 1, represents a particular design procedure. It is evident that not all models can be used in all possible combinations. The rule should be to provide a sensible relation in the levels of advancement of both models. In the text in Fig. 1, reference is made to this aspect [1, 2].

3. HEAT EXPOSURE MODELS

As discussed in the introduction most countries use a fire engineering design in which structural performance is connected to grading of elements of construction in a standard fire test (heat exposure model H_1). Generally the required time of fire duration is not only related to the estimated fire exposure, but is also differentiated with respect to safety considerations relevant to the building in question.

The standard temperature-time relationship according to ISO 834 [3] is given by the following formula (see Fig. 2):

$$T-T_o = 345 \log_{10} (8t + 1) \qquad (3.1)$$

where:
t = time, in minutes
T = furnace temperature at time t, in °C
T_o = furnace temperature at time t = 0, in °C.

The required time of fire duration is usually expressed in multiples of 30 minutes.

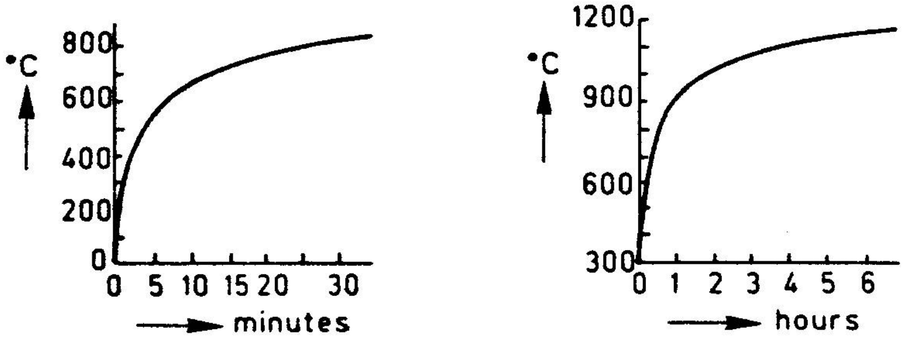


Fig. 2 Standard temperature-time curve [3]

The rise of temperature as a function of time according to ISO 834 and the fire duration are a rough approximation of the real gas-temperature-time curve of a fully developed compartment fire. It is possible to calculate a complete gas-temperature-time curve using heat balance equations (heat exposure model H_3) [4, 5, 6, 7]. The amount of combustible material (fire load), the combustion characteristics of the fire load and the geometrical, ventilation and thermal properties of the fire compartment are the important factors. Safety considerations are related to the determination of the design fire load via a set of partial factors [1, 2]. Fig. 3 exemplifies the result of heat balance calculations for a fully developed compartment fire, with given thermal properties of the compartment and with varying values for the fire load density q and the ventilation factor $A\sqrt{h}/A_t$ [4].
The fire load density q is given by the relationship:

$$q = \frac{1}{A_t} \Sigma \mu_\nu m_\nu H_\nu \ (MJ.m^{-2}) \qquad (3.2)$$

where:
m_ν = total mass of combustible material ν (kg)
H_ν = calorific value of combustible material ν ($MJ.kg^{-1}$)
μ_ν = a fraction between 0 and 1, giving the real degree of combustion for each individual component ν of the fire load, generally assumed equal 1
A_t = total interior area of the surface bounding the fire compartment, including all openings (m^2)

The ventilation factor of the fire compartment is given by the term $A\sqrt{h}/A_t$, in which:

A = total area of door and window openings (m^2)
h = mean value of the heights of the openings, weighted with respect to each individual opening area (m)

The temperature-time curve of a fully developed compartment fire (heat exposure model H_3) must be calculated in principle, for any individual application, from the energy and mass balance equation for the fire load and the fire compartment

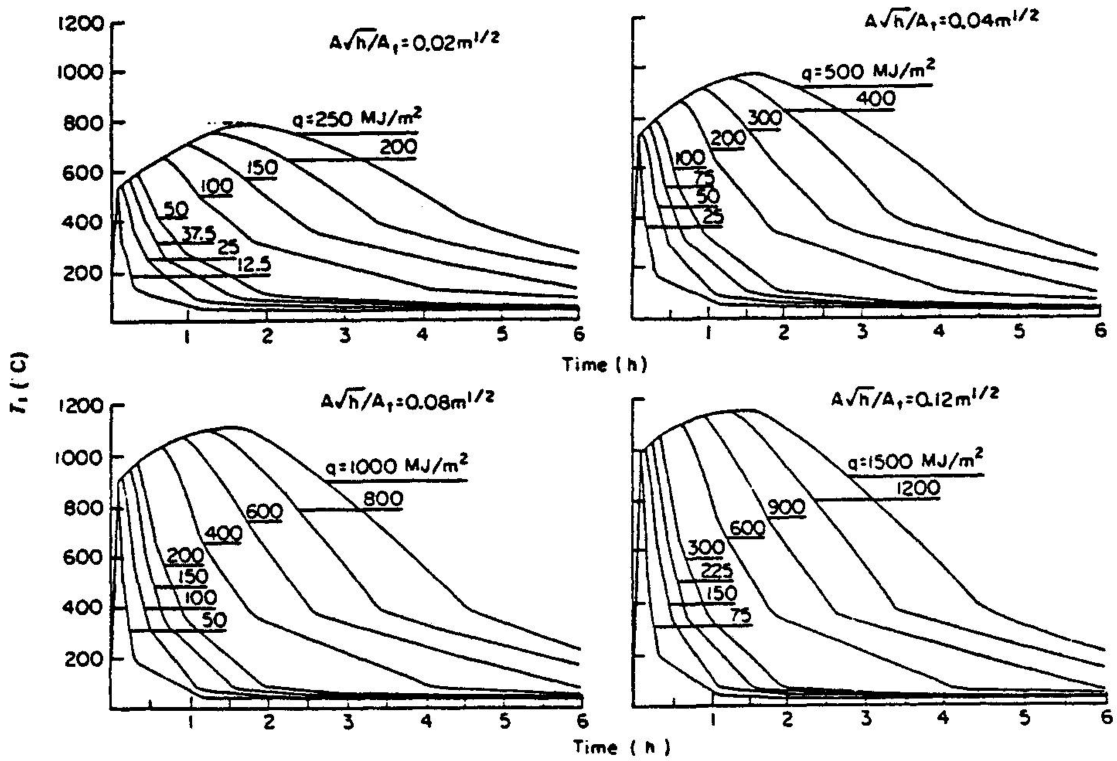


Fig. 3 Gas-temperature-time curves for a complete, fully developed compartment fire with varying values for the fire load density q and the opening factor $A\sqrt{h}/A_t$ according to Pettersson, Magnusson and Thor [4]

in question. For practical applications this requires a computer or a comprehensive set of design charts for different fire loads, ventilation factors and fire compartment characteristics [4]. Moreover, heat exposure model H_3 cannot be used in combination with experimental structural models, which are generally based on the standard temperature-time curve. This is a serious constraint indeed, because for many structural applications, in particular non-load bearing structures like partitions and doors, the fire resistance test even constitutes the only method of verification. So far only the load bearing capacity of steel structures and in a limited sense of concrete structures can be obtained by analytical methods. Therefore, heat exposure model H_2 has been developed, which connects the natural fire, according to heat exposure model H_3 with the standard fire (heat exposure model H_1). The connection between the natural fire and the standard fire comprises a determination of the ultimate state of a representative structural element for a natural fire on one hand and for a thermal exposure according to the standard fire on the other hand. An equivalent time of fire exposure can be defined as that length of the heating period of the standard curve, which gives the same decisive effect on the structural element with respect to failure as the complete process of a natural fire. In a generalized approximate approach, the equivalent time of fire exposure is independent from the type of structural element and follows from the value of the fire load density, the geometry and ventilation characteristics of the fire compartment.
For steel structures, the equivalent time of fire exposure t_e can be expressed by [5]:

$$t_e = 0.067 \frac{q}{(A\sqrt{h}/A_t)^{0.5}} \text{ (min)} \qquad (3.3)$$

Safety considerations relevant to the building in question are related to the equivalent time of fire exposure via a set of partial factors (see chapter 5) [1, 8, 9].

4. STRUCTURAL RESPONSE MODELS

As discussed in the introduction, most countries use a method of verification based on grading of elements of construction in a standard fire resistance test, with fixed heating conditions according to equation (3.1). Because of limited dimensions of furnaces, only relatively small elements can be tested with simplified end-conditions. In the matrix of Fig. 1 the method applies to structural model S_1 and occasionally S_2. Internationally, the standard fire resistance test according to ISO 834 is used very frequently and for many types of structural applications, it constitutes the only way at present for obtaining the information required for a structural fire engineering design. In spite of this, the standard fire resistance test can be seriously criticized. The specification of the test is insufficient in several aspects, such as heat-flow characteristics of furnaces, material properties and imperfections of the specimen, temperature distribution along members and restraint conditions. Thus, repeated tests in the same furnace, not to mention different furnaces, may yield a considerable variation in results. The structural element to be tested is supposed to be modelled with respect to actual conditions expected in the structure. However, deviations from conditions in the actual structure are unavoidable because of the limited dimensions of the furnaces, idealized characteristics of the loading device and insufficiently defined support conditions during the test [10, 11]. An illustration is given in Fig. 4, which shows some results of a correlation test series on composite columns carried out in different laboratories [12].

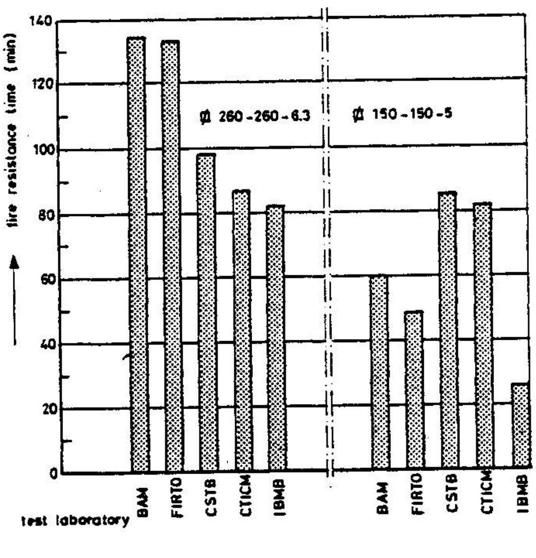


Fig. 4 Some results of fire resistance tests on identical concrete filled hollow steel sections obtained in various test laboratories [12]

Because of these problems and to achieve solutions with a defined and more uniform safety, there is a strong need to move to analytical structural models. Generally these models include two main steps, viz.:

1. A calculation of the temperature distribution within the fire exposed load bearing element or structure during the heating process.
2. A transformation of these temperature distributions to the variation of the load bearing capacity as a function of time in order to examine whether or not the fire exposure will cause a failure of the structural element or structure at the specified loading.

During the last decade, considerable progress has been made in developing analytical design methods for fire exposed load bearing elements and structural assemblies and in making these design methods operational, using design diagrams and tables. This approach is most advanced in the field of steel structures and applies to the structural response models S_1 and S_2 in the matrix of Fig. 1 (see chapter 6). Although in principle, an analytical fire engineering design of structural models of the type S_3 is possible, it may be questioned whether the complexity of the model is justified, as the structural design at room temperature is usually not performed on entire load bearing structures, but is limited to sub-assemblies of the type S_2.

5. PROBABILITY BASED METHODS OF STRUCTURAL FIRE ENGINEERING DESIGN

As discussed in chapter 2, each combination of heat exposure model and structural model represents a particular design procedure. In principle, a differentiated fire engineering design offers a problem-oriented choice for the combination of heat exposure model and structural model as a design method. The final choice may also depend on national preferences, the simplicity of application and on the particular design situation [1, 2].

The design method H_1 - S_1 and occasionally H_1 - S_2, with experimental verification of the fire resistance, corresponds to the vast majority of national building codes. In many countries improved methods based on the heat exposure models H_2 and H_3 [4, 5, 6, 7, 8, 9], have occasionally been used, but, except in Sweden, they are not yet automatically accepted as methods which satisfy the requirements of the building regulations.
In contrast to the acceptance of improved heat exposure models, there is a growing acceptance of design methods H_1 - S_1 with an analytical verification of the fire resistance. In several countries these methods are now being used as an alternative to the standard fire resistance test. Recently the Fire Committee of the European Convention for Constructional Steelwork (ECCS) completed Recommendations providing a reference document for national codes of practice (see chapter 6) [13]. The Recommendations apply to design methods based on heat exposure models H_1 and H_2 and structural response models S_1 and S_2.

Generally, the design criterion in a fire engineering design requires that no limit state is reached during the fire exposure. For a load bearing structure, the design criterion implies that the minimum value of the load bearing capacity $(R_{(t)})$ during the fire exposure shall meet the load effect on the structure (S) i.e.:

$$\min \{R_{(t)}\} - S > 0 \qquad (5.1)$$

In this formula the design criterion is adapted to design methods based on a natural fire, i.e. heat exposure model H_3. For design methods based on the standard temperature-time curve i.e. heat exposure models H_1 and H_2, the design criterion is expressed in a time domain, e.g.:

$$t_{fr} - t_{fd} > 0 \qquad (5.2)$$

where t_{fr} is the time in which the limit state of the structural element is reached, i.e. the fire resistance of the structural element, t_{fd} is the required fire duration specified in the building regulations (heat exposure model H_1) or calculated on the basis of heat exposure model H_2.

In the design methods based on heat exposure model H_2 and H_3, the following probabilistic aspects should be considered (heat exposure model H_1 implicitly includes these aspects).

- Intrinsic randomness of design parameters and properties.
- Model uncertainties of the analytical models for the heat exposure and the structural response.
- Assessment of frequency, such as the probability of occurrence of a large fire, the effect of fire brigade actions, the reliability of sprinklers.
- Safety considerations from both the human and economic point of view such as, the height, volume and occupancy of the building, the availability of escape routes and rescue facilities as well as the consequence of violating a limit state.

Introducing these sources in a probabilistic manner into the design means that they must be expressed in numerical values. The level of the probabilistic analysis may well be limited to a semi-probabilistic approach, in which the aspects mentioned above are clustered and expressed in partial factors and characteristic values are used for action and response effects.
For the design method $H_2 - S_2$ with an analytical structural model, this probabilistic design format reads [1, 2, 8, 9]:

$$\frac{t_f}{\gamma_f} - \gamma_{n1}\,\gamma_{n2}\,\gamma_e\,t_e \geq 0 \qquad (5.3)$$

The structural response model represents the first term of the equation and the heat exposure model the second term.

t_f = analytically determined fire resistance time of a sub-assembly
t_e equivalent time of fire exposure for the fire load and the fire compartment in question
γ_f =partial factor taking into account intrinsic randomness of design parameters and material properties at elevated temperatures, uncertainty in loads and load combinations, as well as uncertainty in the analytical structural response model
γ_e = partial factor taking into account the uncertainty in specifying the fire load, ventilation characteristics of the fire compartment and the thermal properties of the enclosure, as well as uncertainty in the heat exposure model
γ_{n1} = partial factor taking into account the assessment of frequency
γ_{n2} = partial factor taking into account the safety considerations

The partial factors γ follow from statistical data and socio-economic optimization supplemented by engineering judgement [8, 9]. The design can be simplified by using unified γ factors for certain classes of buildings, such as appartment buildings, schools, offices etc.
Finally it should be emphasized that a transition from a purely deterministic classification system to probability based methods of design, including analytical design methods as an alternative to the standard fire resistance test, requires improvement and extension of the concepts outlined, as well as extensive calibration to existing code requirements [1, 2, 8, 9, 14, 15].

6. BEHAVIOUR OF STRUCTURAL STEEL AT FIRE EXPOSURE

The analytical model for the calculation of the load bearing capacity of structural steel exposed to fire includes two steps, i.e.

1. A calculation of the temperature distribution within the structure during the heating process.
2. A transformation of the temperature distribution to the variation of the load bearing capacity as a function of time, in order to examine whether or not the fire exposure will cause a failure at the specified loading.

The design basis will be summarized below and is focussed on simplified models, equivalent to conventional methods of structural design at room temperatures [13].

6.1. Steel temperature as a function of time

The analysis of the temperature distribution within the fire exposed structure during the heating process, may be generally based on the following simplified assumptions:

- constant thermal properties of structural and insulation materials assumed to be the average for the temperature range,
- the steel is assumed to offer no resistance to heat flow and therefore to be at a uniform temperature,
- the resistance to heat flow between the inner surface of the insulation material and the steel is assumed to be zero.

Under these conditions, the temperature distribution in the steel can be calculated with classical one-dimensional heat flow theory [4, 16, 17, 18]. Under the given assumptions, the resistance of unprotected steel members to heat flow is governed only by convection and radiation. The coefficient of heat transfer due to convection from the fire to the exposed surface of the steel member α_c, is considered to be constant with a value: $\alpha_c = 25$ W/m^2 ^oC.
The coefficient of heat transfer due to radiation α_r, is a function of the gas and steel temperatures and can be determined from the Stefan-Bolzmann law of radiation. The resultant emissivity ε_r of the flames, gases and exposed surfaces which appears in this formula, may be considered constant with a value of $\varepsilon_r = 0.5$, giving a conservative solution.
For a fire exposed unprotected steel structure, the energy balance equation gives the following formula for a determination of the steel temperature:

$$\Delta T_s = \frac{\alpha}{c_s \, \rho_s} \cdot \frac{F}{A} \cdot (T_t - T_s) \, \Delta t \; [^oC] \qquad (6.1)$$

in which: $\alpha = \alpha_c + \alpha_r$ [W/m^2 ^oC]
T_t = gas temperature at time t [^oC]
T_s = steel temperature [^oC]
ΔT_s = change of steel temperature during time step Δt
c_s = specific heat of steel [J/kg ^oC]
ρ_s = density of steel [kg/m^3]
F = fire exposed surface per unit length [m]
A = volume of steel per unit length [m^2]

The resistance to heat flow of insulated steel members is governed by convection, radiation and the thermal conductivity of the insulation material. For practical applications however, the influence of convection and radiation can be neglected. Also a distinction is made between lightly insulated members, for which the heat capacity of the insulation material can be neglected, and heavily insulated members, for which the heat capacity of the insulation is taken into account in an approximate way.
For lightly insulated materials, the energy balance equation is:

$$\Delta T_s = \frac{\frac{\lambda}{d}}{c_s \, \rho_s} \cdot \frac{F}{A} \cdot (T_t - T_s) \, \Delta t \; [^oC] \qquad (6.2)$$

in which: λ = thermal conductivity insulation [W/m ^oC]
d = thickness of insulation [m]

For heavily insulated members half of the heat-capacity of the insulation is added to the heat-capacity $c_s \, \rho_s \, A$ of the steel.
In [4] and [13, 17] design tables are given for natural fire exposure and standard fire exposure respectivily.
During deformation at fire exposure, cracks or openings may occur. In order to include these effects for determining the thermal conductivity of the

insulation material, apart from small scale experiments, at least one full scale test on a loaded member must be performed [17, 19].

6.2. Steel properties and structural analysis at elevated temperatures

In general a structure under fire exposure is subjected to a constant load and a temperature increase as a function of time. Depending on the type and thickness of the insulation, the rate of heating can vary. Research reported in [20, 21] has shown that for practical heating rates and for temperatures not over 600 °C, the deformation behaviour under constant load can be considered as independent of the heating rate. Consequently a family of stress-strain relationships for different temperatures must exist, in which the influence of high temperature creep is implicitly included (Fig. 5).

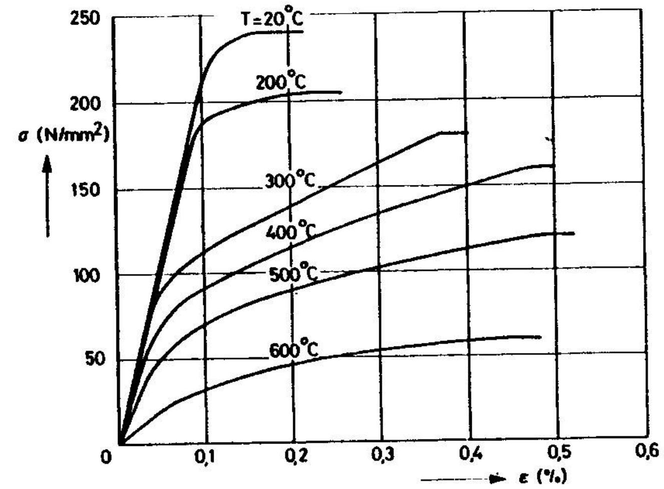


Fig. 5 Stress-strain relationships for Fe 360 at elevated temperatures

The gap between the curves applying to 200 °C and 300 °C is due to so-called "thermally activated flow" [20, 21]. Applying the elementary theory of plasticity, the curved stress-strain diagrams are cut off at certain stress levels. The horizontal plateau is defined as the effective yield stress. In Fig. 6, the effective yield stress variation with steel temperature is given as a fraction of the yield stress at room temperature.

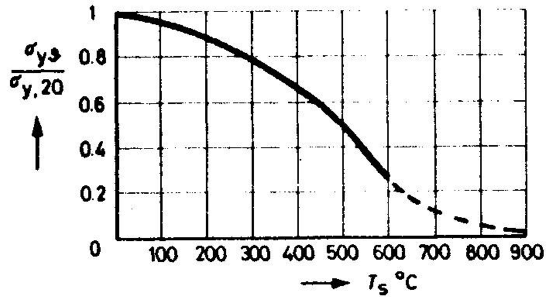


Fig. 6 Effective yield stress $\sigma_{y\nu}$ at elevated temperatures expressed as a fraction of the yield stress at room temperature $\sigma_{y,20}$ (Fe 360 - Fe 510)

The structural analysis of fire exposed structures may be generally based on the following simplified assumptions:

- a time dependent uniform temperature distribution over the height of the cross section and along the members,

- mechanical properties of steel at elevated temperatures which are assumed to be independent of time, i.e. creep effects are included implicitly (fig. 5, 6),
- the load is assumed to be constant and equal to the design load at the service state e.g. dead load + characteristic live loads.

Due to the non-linear stress-strain relationships of steel at elevated temperatures, the linear theory of elasticity cannot be applied and use has to be made of the theory of plasticity.
Two design methods are available, identical to those used in structural analysis at room temperature:

- a limit state design according to the elementary theory of plasticity in those cases where a similar design is allowed at room temperature,
- an incremental elasto-plastic analysis.

The first method is suitable when the limit state at elevated temperatures can be defined by structural collapse, i.e. beams in braced frames. At a given temperature the ultimate load can be calculated from the temperature dependent effective yield stress $\sigma_{y,\nu}$ (Fig. 6). This is illustrated in Fig. 7 [13, 17, 22].

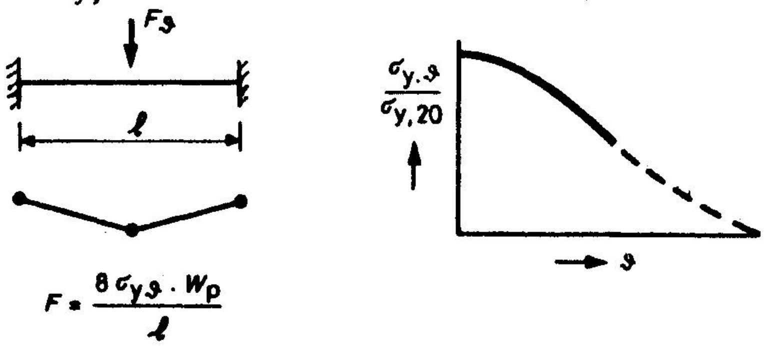


Fig. 7 Structural design at elevated temperatures according to the elementary theory of plasticity

In the European Recommendations for the fire safety of steel structures [13] numerous diagrams and tables are given for the determination of the load-bearing capacity at elevated temperatures of beams, columns and braced frames. Fig. 8 gives an example of such a diagram for unrestrained axially loaded columns, based on tests performed recently in Belgium and some other countries [23].

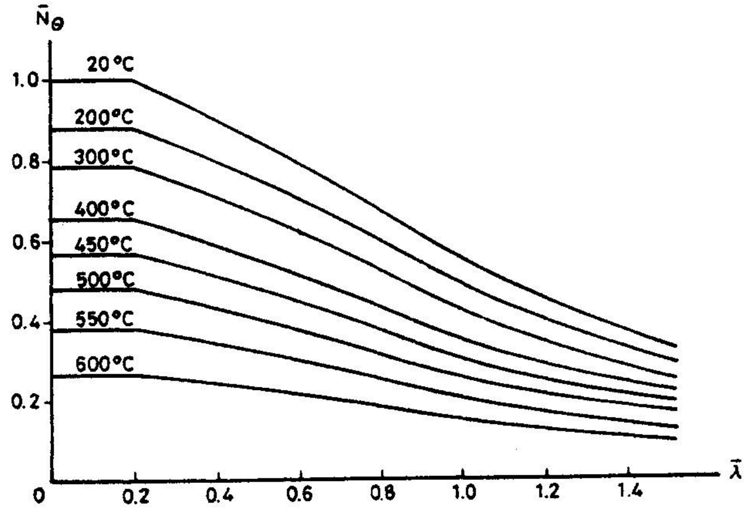


Fig. 8 Relationship between non-dimensional buckling load $\bar{N}_\theta$ and slenderness factor $\bar{\lambda}$ at varying steel temperature T_s for unrestrained axially loaded steel columns [23]

The second method has to be used when the limit state at elevated temperatures is defined by a criterion based on deflections or a rate of deflection. This method must also be applied when geometrically non-linear effects have a significant bearing on the structural behaviour i.e. columns and unbraced frames. At a given temperature, the load-bearing capacity can be determined with the associated stress-strain relationship (Fig. 5), by computing the deflection curve. Fig. 9 gives an illustration. Application of this method usually requires a computer [21].

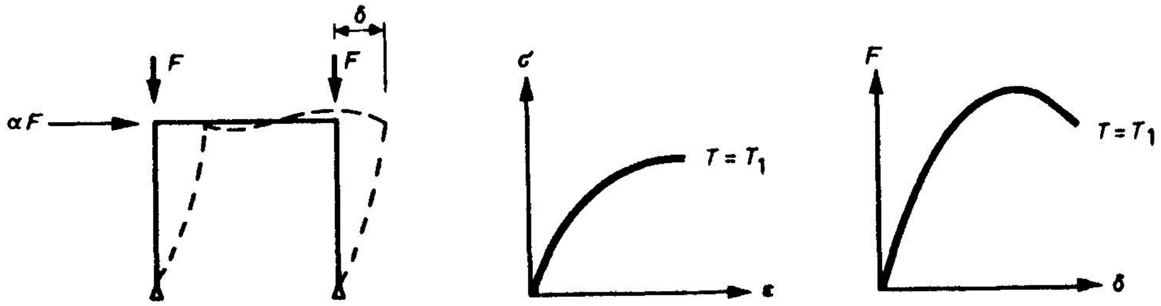


Fig. 9 Structural design at elevated temperatures with an incremental elasto-plastic analysis

This method is particularly used for research purposes, from which simplified design rules can be obtained and implemented in codes for structural fire safety.

ACKNOWLEDGEMENT

International cooperation on the development of new concepts for structural fire engineering design takes place in the Fire Committee of the Conseil International du Bâtiment (CIB/W14) [2]. Design methods for structural steel exposed to fire are coordinated in the Fire Committee of the European Convention for Constructional Steelwork (ECCS-TC3) [5, 13, 24, 25]. The author is grateful for the stimulating discussions and contributions in these committees, which certainly have influenced the contents of this paper.

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Seismic Design of Steel Buildings in Japan

Conception antisismique des bâtiments en acier au Japon

Seismischer Entwurf von Stahlbauten in Japan

Ben KATO
Prof. of Struct. Eng.
University of Tokyo
Tokyo, Japan

Ben Kato, born 1929, received his degree of Dr. of Engineering from the University of Tokyo. He has served as lecturer and associate professor at the University of Tokyo and now occupies the chair of steel structures and welding engineering.

SUMMARY
The basic concept of the new Japanese seismic code is introduced. Then the theoretical and experimental backgrounds of this code are discussed focusing attention on the design of steel structures.

RÉSUMÉ
Les concepts de base des normes antisismiques japonaises sont présentés. Leur provenance, basée à la fois sur l'expérience et la théorie, est analysée du point de vue de la conception des charpentes métalliques.

ZUSAMMENFASSUNG
Das Grundkonzept des neuen japanischen seismischen Codes wird vorgestellt. Die theoretischen und experimentellen Grundlagen dieses Codes, im Hinblick auf den Entwurf von Stahlbauten, werden besprochen.

1. INTRODUCTION

The Japanese national building code for seismic design was revised in June 1981. It took five years for drafting and furthermore took another three years to put the draft into the practical final code,after getting consensus of administrators, practical engineers and researchers.

This paper introduces the basic concept of the new code firstly. Then,the theoretical and experimental backgrounds are discussed focussing on the design of steel structures.

2. BASIC CONCEPT OF THE NEW SEISMIC CODE

Basic concept and structure of the new code are introduced herein,more detailed description is given in reference.1.

2.1 Design criteria

Similar to the design against other loading conditions,two classes of limit states are pertinent to earthquake-resistant building design. They are;(1) the serviceability limit state for a moderate intensity earthquake;and (2) the ultimate limit state for a major earthquake.

-(1)Serviceability limit state design
The structure should be proportioned to resist the moderate earthquake elastically and without excessive lateral deflection so as the building can remain in serviceable condition as soon as the earthquake is over. Moderate earthquakes are expected to occur with a reasonably high probability during the life of a structure. The maximum design spectral acceleration of short-period structures against a moderate earthquake is 0.2g in Japan,where g= the acceleration of gravity.

-(2)Ultimate limit state design
The structure may be permitted to undergo considerable structural damage when it is subjected to a major earthquake. The collapse of the structure and resulting loss of human life,however,must be avoided. A major earthquake is unlikely to occur within the life of a structure,but is used in the design to examine the ultimate structural safety. The maximum design spectral acceleration of a short-period structure in the case of a major earthquake is 1.0g in Japan.

Since the earthquake loading is unique,the definition of load intensity for serviceability limit state is somewhat different from other types of loadings.

2.2 Serviceability limit state design

The lateral seismic shear, ${}_eQ_i$, of the i-th story above the ground level is given as

$${}_eQ_i = {}_eC_i \cdot W_i; \quad {}_eC_i = Z\, R_t\, A_i\, {}_eC_o \tag{1}$$

in which ${}_eC_i$= the lateral seismic shear coefficient of the i-th story for serviceability limit state design; ${}_eC_o$= standard base shear coefficient for serviceability limit state design; W_i=weight of the building above i-th story; Z=seismic hazard zoning coefficient(1.0-0.7);R_t=nondimensional response spectrum(design spectral coefficient) which is determined by the type of subsoil conditions(hard,medium and soft) and fundamental period of the building(T,sec) as illustrated in Fig.1; A_i=lateral shear distribution factor as shown bellow,

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right)\frac{2T}{1+3T} \qquad (2)$$

$\alpha_i = W_i/W$, where W_i is the weight above i-th story and W is the total weight of the building above the ground level.

A structure should be proportioned to be elastic against the lateral forces ${}_eQ_i$ given by Eq.1,and the drift of each story must be less than 1/200 of story height,the value of which can be increased up to 1/120,if non-structural elements are flexible enough to follow-up this magnitude of deformation.

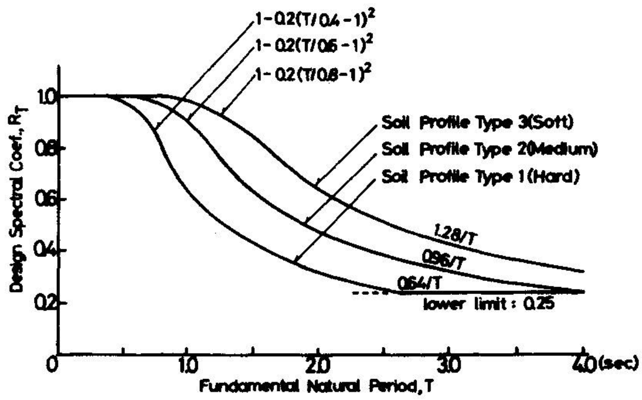

FIG.1 Design spectral coefficient,R_t

2.3 Ultimate limit state design

The lateral seismic shear,${}_uQ_i$,of the i-th story above the ground level is given as

$${}_uQ_i = D_s\, F_{es}\, {}_uC_i\, W_i; \qquad {}_uC_i = Z\, R_t\, A_i\, {}_uC_0 \qquad (3)$$

in which D_s=structural characteristics factor which represents energy dissipating capacity of the building structure related with ductility for each story; F_{es}= shape factor which reflects the adverse effects of eccentricity of stiffness and a drastic change of stiffness along the height; and ${}_uC_0$ = 1.0 = standard base shear coefficient for ultimate limit state design.

2.4 Special provisions

2.4.1 Exemption of ultimate limit state design

In steel buildings not exceeding 31m in height and satisfying the following requirements,ultimate limit state design as specified in 2.3 is not required.

-1.Eccentricity of stiffness and change of stiffness along the height should be negligible and thus F_{es}=1 should be met.

-2.For braced frames,the following increased design seismic shear should be used

$$Q_{bi} = (1 + 0.7\beta)\ {}_eQ_i \qquad (4)$$

in which β = the ratio of lateral shear capacity of diagonal bracings to the total lateral shear capacity of the story.

-3.Joint strength of diagonal bracings should meet the following condition,

$${}_jT_u \geq 1.2\ T_y \qquad (5)$$

in which ${}_jT_u$= ultimate strength of joint of a diagonal bracing and T_y= yield strength of the bracing member.

-4.Width-to-thickness ratios of plate elements of beam-columns and beams shall meet the ductility class I of Table.2 given in 3.2.2.

-5.Strength of beam-to-column connections shall meet the following condition,

$$_jM_u \geq 1.3\ M_y \tag{6}$$

in which $_jM_u$=maximum bending strength of beam-to-column connection and M_y=yield moment of the pertinent beam or column.

2.4.2 Highrise buildings

Design of buildings whose height exceeds 60 meters should be carried out on the basis of time history dynamic analysis for two levels of input earthquake ground motions and the design procedure must be reviewed by the special committee appointed by Minister of Construction.

3. COMMENTARY

The response spectra provide the meaningful measure of the intensity of an earthquake motion. They are expressed on the basis of the following characteristic responses,

Spectral pseudo-velocity response

$$S_v = \left[\int_0^t \ddot{v}_g(\tau)\ \exp[-\xi\ \omega(t-\tau)]\sin\omega(t-\tau)d\tau\right]_{max.} \tag{7}$$

in which v_g=ground displacement; ξ=damping ratio and ω=undamped natural circular frequency.

Spectral displacement

$$S_d = \frac{S_v}{\omega} \tag{8}$$

Spectral acceleration

$$S_a = \omega\ S_v \tag{9}$$

These responses can be applied to the linear elastic structures,and the design criteria for serviceability limit state(elastic limit state) as prescribed in 2.2 can be formulated on the basis of the concept of the response spectra. In fact,$R_t\ {}_eC_o$ in Eq.1 is the nondimensional spectral acceleration response, and related to S_a as $R_t\ {}_eC_o = S_a/g$, in which g is the acceleration of gravity. And the basic structure of this criterion is much the same as other ones specified in many seismically active countries.

On the other hand,the response spectra cannot apply directly to the ultimate limit state design since it involves inelastic deformations. To overcome this difficulty,the design criterion for ultimate limit state is based on energy concept making use the fact that the input energy E into a structure during an earthquake is given as,whether it behaves elastically or not[2,3,4]

$$E = \frac{1}{2}\ M\ S_v^2 \tag{10}$$

in which M=total mass of the structure. Average velocity response spectrum can be approximated by two straight lines as shown in Fig.2. This means that the value of S_v is independent of the fundamental period T for its medium range. Since the fundamental period of a struct-

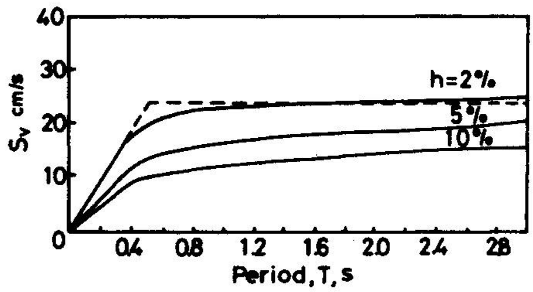

FIG.2 Average velocity response spectrum

ure changes when it is plastified during vibration,above characteristic is very convenient for practical application. On the other hand,S_v changes linearly with T in short period region and the characteristic is uncertain for long period region,therefore,the application of Eq.10 to the structures with very short or very long fundamental periods leaves some questions. This is one reason why the special design procedure is required for high-rise buildings in 2.4.2. Another reasons are to check the damage concentration into a particular story and to check the P-Δ effect.
The followings are theoretical and experimental backgrounds of the formulation of ultimate limit state design criteria.

3.1 Safety criterion

The input earthquake energy into a structure given by Eq.10 is absorbed and dissipated by the elastic strain energy W_e and the cumulative plastic strain energy W_p. For the survival of a structure,the structure's capacity of cumulative plastic energy dissipation ${}_uW_p$ must be greater than the cumulative plastic energy demand,and thus

$$ {}_uW_p \geq W_p = \frac{1}{2} M S_v^2 - W_e \tag{11}$$

This is the criterion to evaluate the safety of a steel structure in the major earthquake.

The elastic strain energy W_e is approximately given as[5]

$$ W_e = \frac{1}{2} M \left(\frac{T}{2\pi} \alpha_1 g\right)^2 \tag{12}$$

in which α_1= yield base shear coefficient.

The earthquake input energy of a multistory building is distributed to each story. If a structure is poorly proportioned,the input energy will concentrate on a particular story. In this sense,it is important to determine the distribution of design shear coefficient along the height so as to develop uniform cumulative plastic deformation at each story. The lateral shear distribution factor A_i given by Eq.2 was found to be suitable one to satisfy this requirement by a series of parametric study[6,7]. Through this study,the information on the distribution of plastic works done by each story was also obtained,therefore, the safety of a structure can be examined at any one story. From the viewpoint of practical design,however,it is convenient to determine the required yield base shear coefficient by carrying out the safety check by Eq.11 at the first story and then to determine the yield shear coefficient for upper stories in accordance with Eq.2.
The ratio,a_1,of the plastic work by the whole structure to that by the first story obtained from above study is

$$ a_1 = \frac{{}_uW_p}{{}_uW_{p1}} = \sum_{i=1}^{N} s_i \, d_i^{-12} \tag{13}$$

in which ${}_uW_{p1}$=plastic work done by the first story; s_i=energy distribution ratio at i-th story relating to the distribution of mass,stiffness and yield shear coefficient of structure and d_i=coefficient at i-th story reflecting an inevitable discrepancy between the optimum and actual yield shear coefficient distribution.

The hysteretic shear force-deflection relationship of a story is related to the monotonic loading curve; thus, the cumulative plastic work is also related to the plastic work under monotonic loading. If this equivalent monotonic loading curve is depicted by Fig.3, in which Q_{y1}=yield base shear force; δ_{y1}=first story yield deformation; δ_{m1}=critical deformation, and $\eta_1=(\delta_{m1}-\delta_{y1})/\delta_{y1}$=critical cumulative ductility ratio, the capacity of cumulative plastic work by the first story in the two directions is

$$_uW_{p1}=2Q_{y1}\,\eta_1\,\delta_{y1}=4W_e c_1\eta_1 \tag{14}$$

in which $c_1=k_{eq}/k_1$; $k_{eq}=4\pi^2M/T^2$=equivalent spring constant of the whole structure, and k_1=spring constant of the first story.

Combining Eqs.11,12,13 and 14, the required yield shear coefficient of the first story, α_1, is defined by the plastic deformation capacity, η_1, and the intensity of the earthquake, $S_a=(2\pi/T)S_v$,

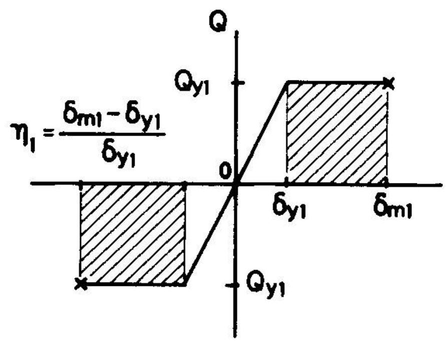

FIG.3 Cumulative ductility ratio

$$\alpha_1 g \geq \frac{1}{\sqrt{1+4c_1a_1\eta_1}} \tag{15}$$

Eq.15 can be rewritten as

$$Q_{y1}\geq D_s\,Q_e;\qquad D_s=\frac{1}{\sqrt{1+4c_1a_1\eta_1}} \tag{16}$$

in which $Q_{y1}=\alpha_1 gM$=required yield base shear strength, and $Q_e=S_aM$= elastic maximum shear force corresponding to the spectral acceleration response S_a. Thus the basic skeleton of the ultimate limit state design given by Eq.3 was derived.

3.2 Plastic deformation capacity of steel frame

The evaluation of critical cumulative ductility ratio, η, is necessary to determine the structural characteristic factor D_s. η can be determined by evaluating the plastic deformation capacity of steel frames. Failure of the steel frame under load reversals occurs when the cumulative plastic deformation in one direction reaches the capacity of plastic deformation under monotonic loading. And the plastic deformation capacity of a frame under monotonic loading is governed by the local buckling, flexural torsional buckling and breaking of its member elements.

3.2.1 Frame ductility and member ductilities

As a feasible approach, multibay, multistory frame was reduced into a linkage of unit frames, and the deformation capacity of the unit frame for each story was evaluated on the basis of member ductilities. The deformation of a story unit-frame consists of deformations of columns, beams and joint panels. In general, it is likely that all these elements be plastified at the ultimate state of the frame. However, to develop a simple design rule, it was assumed that one member element of the unit frame(columns or beams) contribute to the plastic deformation of the frame. Furthermore, the effects of plastic shear deformation of joint-panels and the apparent increase of deformability due to Baushinger's effect were considered on the basis of experimental results, and finally an empirical formula that relates the frame ductility on average to the ductility of individual members was obtained as

$$\eta = \frac{2}{3}\,{}_{I}\eta + 2.0 \tag{17}$$

in which ${}_{I}\eta$ = ductility ratio of columns or beams, whichever is smaller.

3.2.2 Ductility ratios of individual members

The slenderness of beams and columns is limited as follows;

For columns: $\lambda_y \leq 70$ (for grade SS41 steel and SM50 steel)

For beams: grade SS41 steel, $\lambda_y \leq 150 + 20n$

grade SM50 steel, $\lambda y \leq 130 + 20n$

in which λ_y=slenderness ratio of columns and beams with respect to weak axis; and n=number of equally spaced stiffening members. Nominal yield stress of SS41 is 235 MPa and that of SM50 is 324 MPa.

Under these limitations, steel members fail by the local buckling of plate elements of their sections. Based on a large number of laboratory tests, the rotational ductility ratio of members with H-sections, box-sections, and circular hollow sections was evaluated in terms of the width-to-thickness ratio (diameter-to-thickness ratio) and the axial stress[8].

The allowable rotational ductilities of members, ${}_{I}\eta$, are categorized into three classes considering the convenience of the common design practice, and the corresponding ductility ratios of story frame, η, are calculated by Eq.17, as shown in Table 1. And on the basis of the mentioned study, the limiting width-to-thickness ratios corresponding to each ductility class were determined for various shapes with different dimensions and steel grades, as shown in Table 2. Detailed discussions of 3.1 and 3.2 are given in references [9] and [8] respectively.

Table 1 Member and frame ductility ratio

Ductility ratio	Ductility class		
	I	II	III
${}_{I}\eta$	6.0	1.5	0
η	6.0	3.0	2.0

Table 2 d/t(D/t) ratio limitation for each ductility class

Member	Section	Nominal yield stress, MPa	Width-to-thickness ratio, Ductility class I	II	III
Column	H-shaped flange	235	10	11	16
		324	8	10	13
Column	H-shaped web	235	43	43	48
		324	37	37	41
Column	Box-shaped	235	33	37	48
		324	27	31	41
Column	Circular tube	235	50	70	100
		324	36	50	73
Beam	H-shaped flange	235	9	11	16
		324	8	9	13
Beam	H-shaped web	235	60	65	71
		324	50	55	61

Note; b=width; D=diameter; and t= wall thickness

3.3 Special provision

In 2.4.1, it is stated that, if the height of a steel building does not exceed 31m and if the structural elements satisfy the prescribed requirements, ultimate limit state design is exempted. The prescribed requirements are enough to guarantee the structure for exhibiting the class I ductility(η=6) in Table 1. And introducing this value of η into Eq.16, the D_s-values are obtained to be 0.25-0.3 depending to a_1 and c_1 values.

And if D_s=0.3 is introduced into Eq.3 assuming that F_{es}=1.0,

$$ {}_uQ_i = 0.3\ Z\ R_t\ A_i\ W_i \tag{18} $$

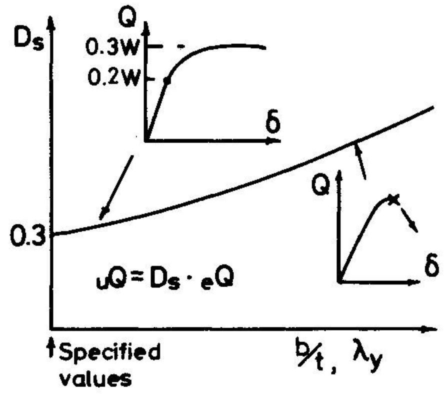

FIG.4 $(b/t,\lambda_y)$-D_s relation

On the other hand, Eq.1 for serviceability limit state is rewritten as

$$ {}_eQ_i = 0.2\ Z\ R_t\ A_i\ W_i \tag{19} $$

Comparing Eq.18 and Eq.19, it can be seen that the ultimate limit state design becomes unnecessary if ${}_uQ_i/{}_eQ_i \geq 1.5$. In usual rigid frames, the ultimate strength, ${}_uQ_i$, is larger than 1.5 times the elastic limit strength, ${}_eQ_i$, due to the effects of moment redistributions and of the increase of bending moment of individual members. The situation is illustrated in Fig.4. This is the rationale of this provision.

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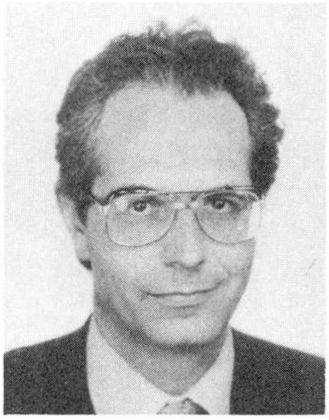
ECCS Approach for the Design of Steel Structures to Resist Earthquakes

L'approche de la CECM pour la conception de structures métalliques anti-sismiques

Lösungsvorschlag der EKS zum Entwurf von Stahltragwerken unter Erdbebenlast

Giulio BALLIO
Professor
Politecnico
Milan, Italy



Giulio Ballio, born 1940, is full Professor of Steel Structures at the Politecnic of Milan. He was chairman of ECCS TC 13-Earthquakes and now he chairs ECCS-TC 1 Safety and Loadings. His professional activity deals with the design of both concrete and steel structures.

SUMMARY

In the last three years the European Convention for Constructional Steelwork (ECCS) has been considering the problems arising in the seismic design of steel structures. In this paper the type of approach followed is highlighted, some results are reported, and future trends and research needs are discussed.

RÉSUMÉ

Pendant les derniers trois ans, la Convention Européenne de la Construction Métallique (CECM) a dédié son attention aux problèmes concernant le sécurité des constructions en acier en zone sismique. Dans cet article on présente les méthodes, utilisées et quelques résultats de ce travail. On indique les études futures et les recherches nécessaires pour compléter l'analyse du problème.

ZUSAMMENFASSUNG

Während der letzten drei Jahre hat sich die Europäische Konvention für Stahlbau (EKS) eingehend mit den Problemen des seismischen Entwurfs von Stahlkonstruktionen befasst. In diesem Vortrag wird ein möglicher Lösungsweg durchleuchtet. Es werden einige Ergebnisse aufgezeigt und auf Zukunftstendenzen und notwendige Forschungen hingewiesen.

1. SEISMIC ACTIONS AND CODES FORMAT

Two different criteria may be found in current codes in order to state seismic actions.

a - The response spectrum is correlated to a reduced value of ground acceleration [1,2,3], approximately the 10-15 % of the expected peak value during a strong earthquake. The structure must be checked at the elastic limit but large plastic deformations may occur during a seismic event. If strictly applied, this approach should lead only to the design of structures with an high level of ductility as frames with rigid joints, braced frames and eccentric bracings. It may not cover some typical european structures as truss bracings, widely adopted for low rise apartment hauses, and isolated columns as commonly used in mill buildings.

b - The response spectrum is correlated to a realistic value of ground acceleration [4,5,6]. Thus the response spectrum is transformed into a design spectrum reducing its values by a factor $q > 1$, the so called "structural behaviour factor". The factor q takes into account the elastic plastic behaviour of the structure, the ductility resources of structural elements and their joints. Such an approach may allow the use of structures with limited resources of ductility, provided that greater values of seismic actions are assumed in the design.

The Eurocode n. 8 - Common Unified Rules for Structures in Seismic Regions, recently issued by the Commission of the European Communities [7] states the design spectrum

$$C(T) = A\,R(T)/q \qquad (1)$$

where:

C(T) is the value of the design spectrum at the period T;

A is the design value of the ground acceleration; depending on the degree of local seismic activity, suggested values of A are between 0.15 and 0.35 g.

R(T) is the value of the normalized design spectrum. It depends on the soil nature and it is stated on the basis of 5% of damping ratio as from fig. 1;

q is the behaviour factor. The Eurocode states: "This parameter takes into account the energy dissipation capacity of a ductile responce. The values of the parameter q depend on the basis of classification of structural system according to ductility levels".

With regard to the above appoach steel structures may be distinguished into two main categories:

- non dissipative structures ($q=1$) designed to withstand seismic actions, and remain in the elastic range.
- dissipative structures ($q>1$) designed in such a way that, during a seismic event, some of their parts (dissipative zones) may move out of the elastic range in order to dissipate energy by mean of a ductile hysteretical behaviour.

Non dissipative structures do not need the ductile behaviour of members and joints to be taken into accont. Dissipative zones of dissipative structures must be designed according to some limitations for joints, slenderness and b/t ratios.

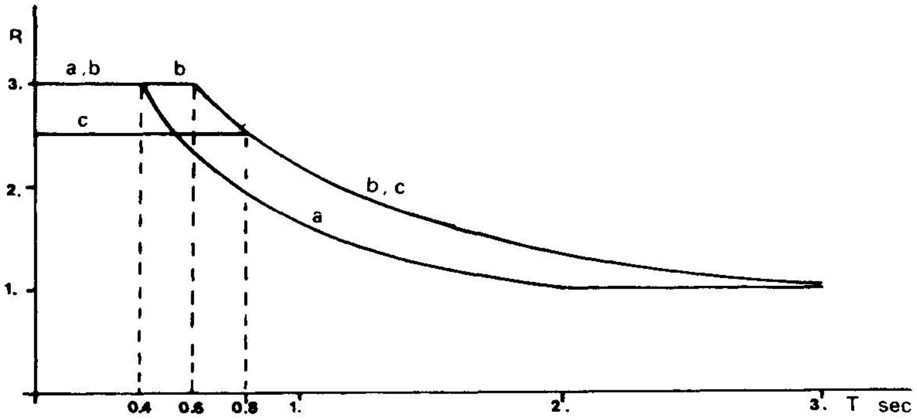


Fig.1 Design spectra as from Eurocode

Tentative values of q factors, as stated by the not jet issued Part III of Eurocode n. 8, are as follows:

- for frames, braced frames, eccentric frames, cross bracings provided that 2nd ordes effects may be disregarded q=4 m
- for the abore structural system but with relevant 2nd order effects q =3 m
- for cantilever structures q =2 m

where m is the ratio between the multiplier of the design loads corresponding to the attainment of the collapse and the multiplier of the design loads corresponding to the attainment of yielding in the most stressed fiber.
Of course for each structural system limitations are given. For example, frames must be designed in order to have dissipative zones in beam elements and not in columns. In eccentric bracings, dissipative zones must be considered in the girders and not in the diagonals. In truss cross bracings tension diagonal members only may be considered active in withstanding lateral forces; their slenderness is limited to 1.5 $\sqrt{(E/f_y)}$.

2 - THE ASSESSMENT OF BEHAVIOUR FACTOR

From formula (1) the following statement can be derived.
" A correct definition of the values of behaviour factor q"
" is foundamental for a reliable and economic design."

Thus the researchers must join their forces in order to state correct values for q factors.
From a theoretical point of vew the parameter q corresponds to the ratio between the seismic intensity (in the sense of the peak value A) which cause the collapse of the structure and the attainment of the elastic limit state . In other terms let us suppose that a structure attains its elastic limit state when subjected to a seismic event (accelerogram) with a peak value A/q. If we scale the accelerogram up to the peak value A, plastic deformations will occur but their values will not exceed the maximum ones consistent with the integrity of the structure.

Thus the following item are necessary in order to state q :
- design the structure at the elastic limit state for a given seismic action (accelerogram with peak value A_o)
- define the plastic limit deformations at critical sections
- increase the seismic action (A/A_o) and predict the elastic plastic behaviour until the limits of plastic deformations are reached for the value A_u
- define $q=A_u/A_o$

It is self evident that this procedure does not lead to practical results. In fact:
- it is applicable only to a well defined structure
- if the structure is designed and methods for predicting elastic plastic behaviour are available, it is not worth while to assess q. The structure may be checked by non linear analysis.

From a practical point of view the problem of assessing q must be semplified. One way may be as follows:
- state the q values depending on structural systems (frames, bracings, inverted pendulum, ect) together with the local demand of ductility
- provide a ductility greater than the demanded one.

In order to accomplish the first step a method independant from the definition of a limiting plastic deformation is needed. A possible approach is as follows.

Let us imagine that an engineer have to design two anologous structures in two different sites with two different codes having the same format. In the site n.1 the code n.1 states a ground acceleration A_1 and a behaviour factor q_1. Let be A_2, q_2 the values for the same quantities in site n. 2. If $A_1/q_1 = A_2/q_2$ the design spectra (1) are equal for both sites and thus the same identical structure is well suited for both sites. Let be v_d the value of the displacement at the elastic limit state of a meaningful point of the structure.

When a seismic event will occur the behaviour of the two structures will not be the same. If the assumption of the ductility factor theory are accomplished [8,9], the displacements will be $v_{A1} = q_1 v_d$; $v_{A2} = q_2 v_d$.
Thus the following statements hold:
- two structure are identical and have the same design displacement v_d if $A /A_1 = q /q_1$
- if the ductility factor theory is valid then $v_A /v_d = q$

Assume an accelerogram with a peak value A and design the structure at the elastic limit state assuming A_1 and q_1. Let v_d be the value of the displacement of a meaningful point. Increase the value A of the peak value and evaluate the maximum value v_A of the displacement. Three patterns are possible (fig.2) Pattern "a" corresponds to a behaviour in compliance with the results of the ductility factor theory. Pattern "b" shows an unsafe behaviour because everywhere $v_A > q\, v_d$ Pattern "c" presents a first safe range $(v_A < qv_d)$ followed by an unsafe one. The values of $q=v_A/v_d$ for which the ductility factor theory is accomplished may be choosen as q values for the structure and $v_A/v_d = q$ represents the ductility overall demand of the structure. The above method was used for assessing q factors for columns of mill buildings [10].

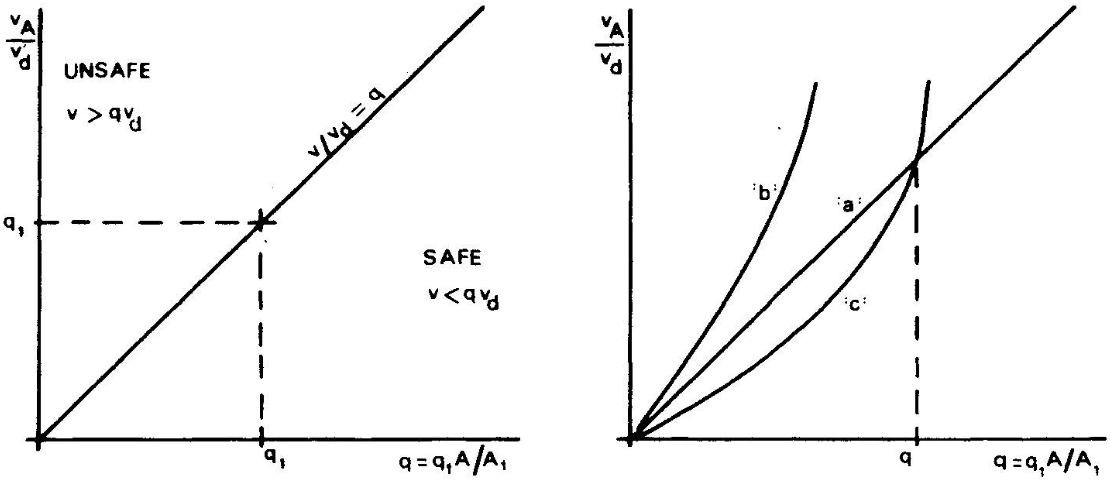


Fig. 2 Determination of the q factor

3 - TESTING PROCEDURES

Numerical models and non linear dynamic analysis are necessary for assessing q values and ductility demand. On the other hand it seems compulsory to perform experimental tests in order to:

- check the correctness of the numerical models
- control the possibility of providing a ductility not less than demanded one.

Shaking table tests are surely the closest to the reality. On the other hand they need very high investments and menagement costs. Thus dynamic tests appear more suited for giving the final proof of the reliability of a structure rather than for appointing a structural system or for comparing different structural solutions.
For the above reasons ECCS pointed its attention to static cycling tests and drafted a recommended procedure [11,12] in order to perform such tests. The major points of this proposal are:

- the choice to impose at each cycle the value of the displacement rather than that of the applied force.
- the definition of various parameters that may characterize the structural behaviour of the speciman (ductility, full ductility, rigidity, maximum load, energy).
- the care of looking at possible deteriorating behaviour imponing three cycles for each value of imposed displacement.
- the criteria for determining the end of the test.

The purpose of this procedure is to standardize the tests in order to produce results that may be compared each other and with the ones of numerical models . As an example in fig.3 are represented the experimental results of a full scale test of a cross bracing .

In fig. 4 the experimental patterns are compared with the results of the numerical simulation of the test by mean of a finite element model [13].

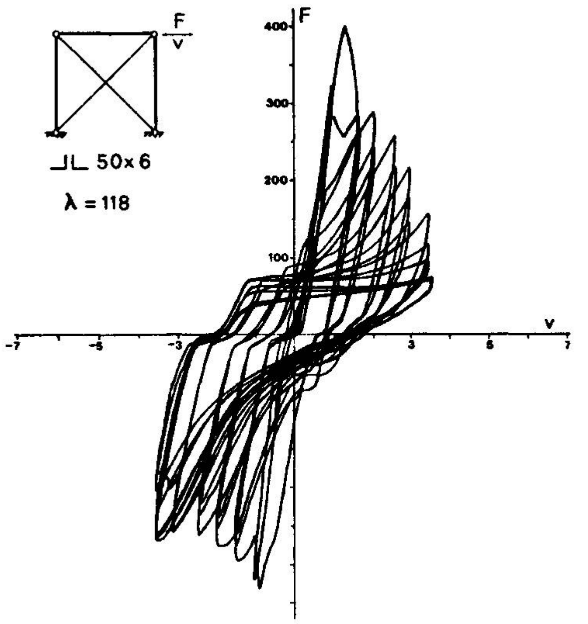


Fig. 3 Experimental test on a cross section truss bracing F (KN) - v (cm)

4 - RESEARCH NEEDS

Both numerical and experimental studies are necessary in order to prove the reliability of most common european steel structures in seismic zones.

a - It is necessary to assess the structure coefficient q for both framed and braced structures with different height (one, four and eight floors are typical for civil buildings).
Similar studies for composite structures are also useful as well as for the most common shapes of industrial buildings.

b - Probabilistic studies are necessary in order to state loading combination for industrial buildings with heavy cranes in seismic zones.

c - Expansion joints, if any, must be much larger in aseismic structures. This condition may give some problems in designing long structures for industrial plants and may suggest to avoid expansion joints even if neglecting temperature effects. Non linear studies looking for a good compromise between temperature effects at serviceability limit states and seismic forces at ultimate limit states must be performed.

d - Ductility of structural elements and connections must be experimentally assessed.Width to thickness limiting ratios in order to avoid local instability for elastic and plastic design are well known, if loads are monothonically increasing. It is necessary to state limits to b/t ratios also when cycling loads may occur in order to allow or forbid the use of cold formed profiles for aseismic structures.

e -The semi-rigid joints are developping for their economic convenience. Their suitability in seismic zone is still undemostrated as well as the benefits of slipping in bolted connections. Test on models, subassemblages and full-scale structures are needed.

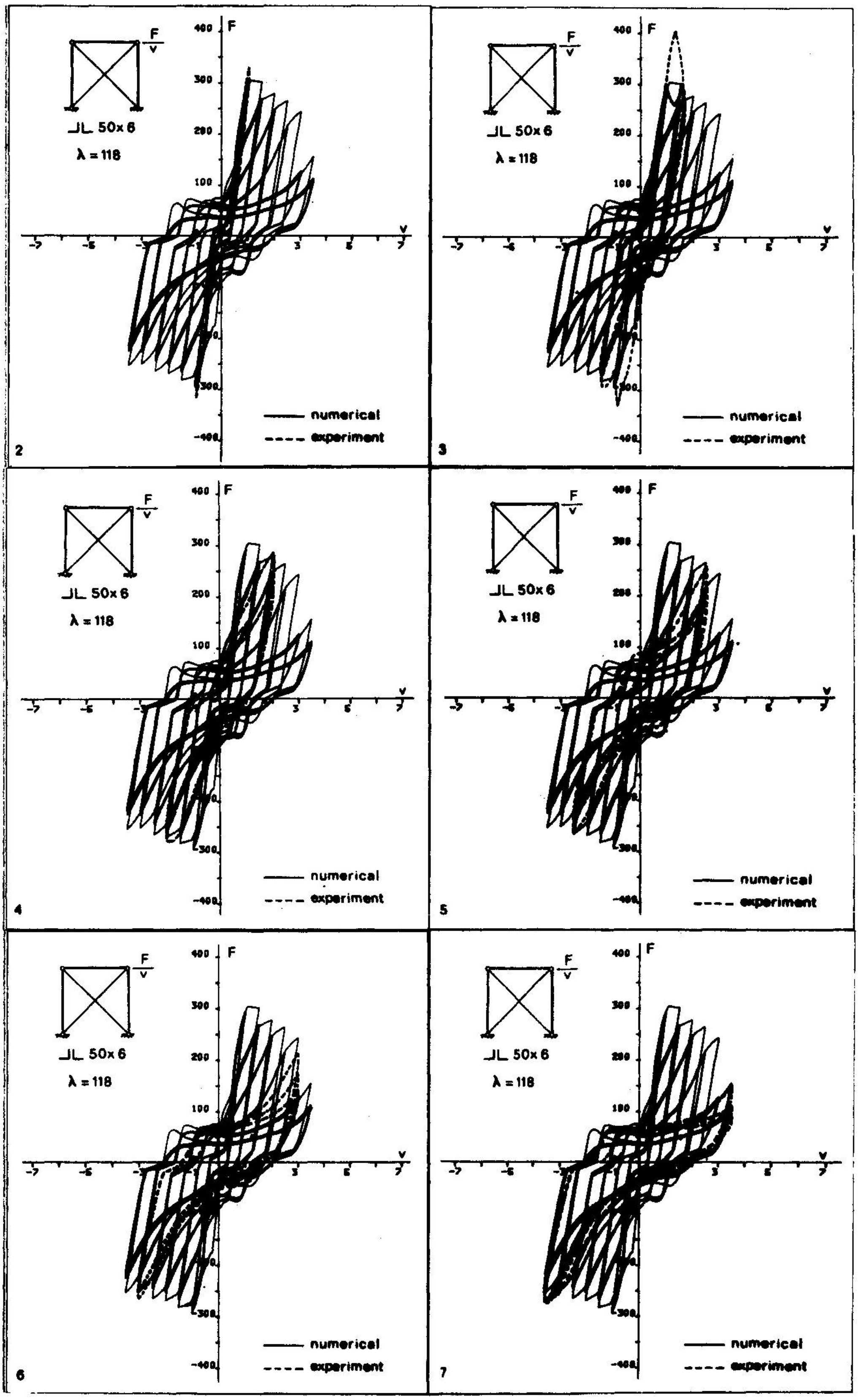


Fig. 4 - Numerical results on the bracing of Fig. 3

At present the efforts of ECCS Working Group WG 1.3, are mainly devoted to points a), d), e). They are mainly supported by researchers of Aachen (D), Liege (B), Milan (I), Napoli (I), Rennes (F), but it is hopeful that in future more and more studies will be performed, in order to make deeper the knowledge of steel structure behaviour in seismic zones.

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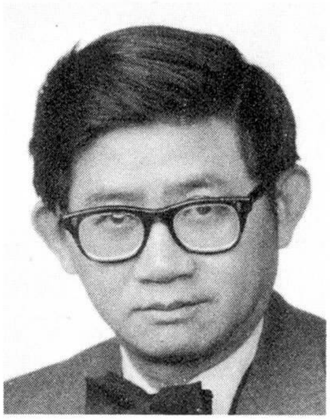
Ductility and Fracture of Joints with Panel Zone Deformation

Ductilité et mode de rupture d'assemblages avec panneau de renfort d'âme

Verformbarkeit und Bruch von Rahmenknoten mit Stegverformung

Le-Wu LU
Prof. of Civil Eng.
Lehigh University
Bethlehem, PA, USA

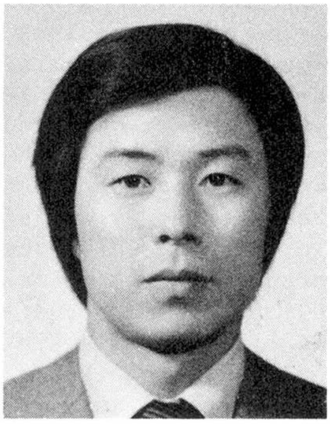


Le-Wu Lu, born 1933, received his BS, MS and PhD degrees in Civil Engineering from the National Taiwan University, the Iowa State University, and Lehigh University, respectively. He has been actively engaged in research in steel, reinforced concrete and composite structures, and in earthquake response of buildings.

Roger G. SLUTTER
Prof. of Civil Eng.
Lehigh University
Bethlehem, PA, USA

Roger G. Slutter, born 1931, received his BS, MS and PhD degrees in Civil Engineering from Lehigh University. He has been actively engaged in research and testing in many aspects of structural engineering and related fields.

Seung J. LEE
Research Assistant
Lehigh University
Bethlehem, PA, USA



Seung J. Lee, born 1951, received his BS degree in Architecture from the Seoul National University and MS degree in Civil Engineering from the Ohio State University. For two years, he was a structural engineer with the Hyun-Dai Construction Co. in Korea.

SUMMARY
An experimental study of the inelastic behavior of beam-to-column joints with panel zone deformation has been carried out and selected results are presented. The factors examined include shear reinforcement of column web, horizontal stiffeners, and composite beam action.

RÉSUMÉ
Une étude expérimentale du comportement inélastique d'assemblages poutres-colonnes avec renforts d'âme a été menée, et les résultats intéressants en sont présentés. Les paramètres examinés sont le renforcement de l'âme de la colonne, les raidisseurs horizontaux et l'effet mixte de la poutre.

ZUSAMMENFASSUNG
Es werden ausgewählte Ergebnisse einer experimentellen Untersuchung über das unelastische Verhalten von Rahmenknoten mit Stegschubverformungen dargestellt (mehrstöckige Rahmen). Der Einfluss der folgenden Parameter wurde untersucht: Schubverstärkung des Stützenstegs, horizontale Aussteifungen und Verbundwirkung zwischen Stahlträger und Betondecke.

1. INTRODUCTION

Building structures are usually designed to satisfy both the serviceability and the strength requirements, a majority of which are specified in applicable codes. If a building is to be built in a seismic region, the overriding design concern is the effect of earthquake. The design practice in the U.S. requires that attention be given to such problems as (1) story drift at the code level earthquake forces, (2) stresses in members under working gravity load and code level earthquake forces (must be less than the code allowable stresses), and (3) response of the structure during a severe earthquake. The last problem requires a careful consideration of ductility and energy absorption capacity of the critical structural elements and of the overall structure.

A structural system that has been widely used in building construction and has performed reasonably well in laboratory testing and during actual earthquakes is the moment-resistant steel frame. The system has good energy absorption capacity, but its stiffness against drift is not high. In designing a moment-resistant frame, it is often necessary to use girders that are considerably larger than those required to satisfy the allowable stress criteria in order to control drift. At the code seismic force level, the stresses in these girders can therefore be substantially less than the allowable values. However, when such a frame is subjected to a major earthquake and is assumed to remain elastic, the lateral forces generated could be several times greater than the code forces. Inelastic action must therefore take place in the highly stressed regions of the structure. One such region is at the ends of the beams, where plastic hinges may form if the weak-beam, strong-column concept is followed in the design and if the joints are capable of transmitting the full plastic moment of the beams.* To satisfy the latter condition, the panel zone of the joint is often strengthened with shear reinforcement such as doubler plates. This increases, sometimes substantially, the fabrication cost. Some structural engineers therefore ask the question: If the girder is sized to meet a drift limitation, is it necessary to design the joint and the connection to develop the full plastic moment of the beam? The Uniform Building Code [1] gives the following guidelines:

> Connections: Each beam or girder moment connection to a column shall be capable of developing in the beam the full plastic capacity of the beam or girder.
>
> Exception: The connection need not develop the full plastic capacity of the beam or girder if it can be shown that adequate ductile joint displacement capacity is provided with a lesser connection.

The above "exception" implies that it is permissible to utilize the inelastic action of the panel zone of the joint to dissipate part of the energy input during an earthquake. The amount of inelastic deformation required of the joints is related to the characteristics of the earthquake ground motion and the properties of the frame. A complete inelastic seismic response analysis is necessary in order to determine the inelastic joint deformation and to evaluate overall performance of the structure. However, before such an analysis can be performed, the behavior of joints with panel zone deformation must be well understood and is properly represented by analytical models.

Among the various factors that affect the behavior of the panel zone, the following are considered to be significant: (1) the amount of shear reinforcement, (2) the presence or absence of horizontal stiffeners (or continuity plates),

*In this paper, a joint is defined as the entire assemblage at the intersection of the members, and a connection is only those elements that connect the member to the joint.

and (3) the details employeed in welding the shear reinforcement and stiffeners. Another problem that has received considerable recent attention is the effect of composite action of girders on joint and panel zone behavior. This is a complex problem, especially when the joint is subjected simultaneously to both positive and negative bending moments.

These problems have been studied in an experimental investigation carried out recently at the Fritz Engineering Laboratory of Lehigh University. The emphasis of the investigation is on the inelastic deformation capacity of the panel zone and the failure mode of the joint under cyclic loading.

2. DESCRIPTION OF TEST SPECIMENS

Three series of girder-to-column joints have been tested. The first series included four full-scale interior joints, three having shear reinforcement in the form of doubler plate and one reinforced. For the three specimens with shear reinforcement, the details of welding the doubler plate to the column varied. The second series, also included four interior joint specimens, examined the effect of horizontal stiffeners on panel zone deformation. The third series studied the behavior of both interior and exterior joints with composite girders. In this series three full-scale specimens, all without shear stiffening, were subjected to cycles of repeated and reversed loading until failure. In this paper, the results of four selected test specimens, two from the first series and one each from the second and third series, are presented and compared with reference to the effects of (1) shear reinforcement, (2) horizontal stiffener, and (3) composite girder action.

All the test joints were made of A36 steel with a nominal yield stress of 250 MPa. The girder flanges were fully welded to the column and the web was bolted to a connection plate with ASTM A325 bolts. The girders were sized to provide sufficient flexural and shear strength to force severe yielding to occur in the panel zone and its boundary elements when no shear reinforcement was added. The web connection was designed to carry all the vertical shear. The three bare steel specimens which were designated as Joints A, B and C, had the same general dimensions and member sizes, as shown in Fig. 1. The composite joint was designated as Joint D, the details of which are given in Fig. 2.

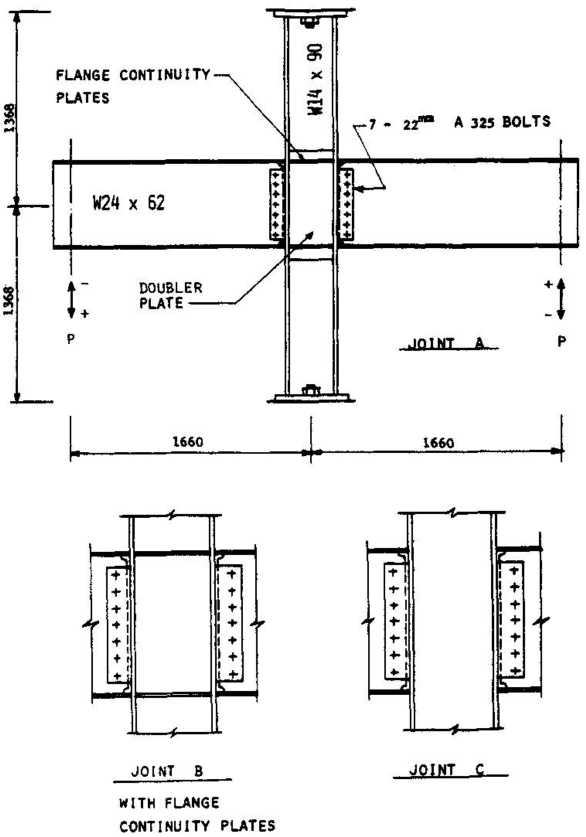


Fig. 1 Dimensions and Details of Joints A, B and C

2.1 Joint A

This was the only joint that was reinforced by both doubler plate and continuity plates. The doubler plate was 12.7 mm (1/2 inch) thick and had a nominal yield stress of 345 MPa. It was welded to the column by fillet welds. This plate together with the web of the column was sufficient to resist the shear transmitted to the joint when plastic hinges formed in both girders. The calculation was based on a shear yield stress of $0.68\sigma_y$ not the von Mises yield stress of $0.58\sigma_y$. (See Ref. 2 for an explanation of the selection of the yield stress.)

2.2 Joint B

This joint was identical to Joint A except that no doubler plate was provided. The joint ductility was expected to be due largely to shear yielding of the panel zone.

2.3 Joint C

Neither doubler plate nor continuity plates were provided in this joint. The results of this test can be compared directly with those of Joint B to evaluate the effect of continuity plates.

2.4 Joint D

This specimen represented an interior joint of a six-story, two-bay prototype test building. The composite slab was cast on a metal deck which was connected to the girder by headed shear studs. The concrete was lightweight with a 28-day compressive strength of about 34 MPa (5000 psi). Although the member sizes of this specimen were not the same as those of the other joints, a qualitative study of the effect can be made in terms of strength and panel zone deformation capacity.

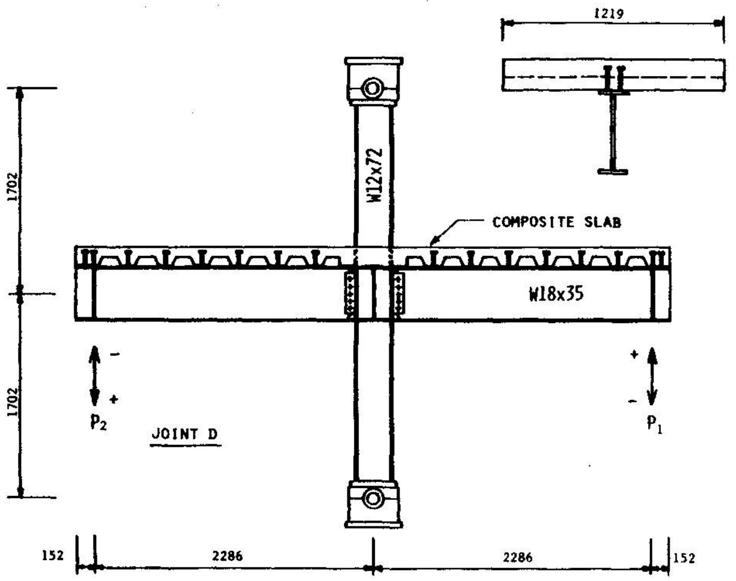


Fig. 2 Dimensions and Details of Joint D

3. EXPERIMENTAL BEHAVIOR AND RESULTS

3.1 Test Procedure

The specimens were tested by repeatedly applying loads in opposite directions to the beams. The direction of each load was also reversed. For Joints A, B and C, the testing was controlled by panel zone shear deformation, except at the early stage when load control was used. The panel zone deformation was measured either by a diagonal gage or by rotation gages attached to the column web. For Joint D, the vertical deflections at the load points were used as the control, and the deflections of the four corners of the panel zone were measured independently. The measured deflections were then converted to panel zone rotation.

3.2 Joint A

In testing the specimen, load increments of 45 kN per beam were used until the panel zone deformation reached approximately 1.0%. The remainder of each cycle was achieved by loading until the diagonal cycle gage indicated increments of approximately 0.5% additional rotation. The loading was continued up to a maximum panel zone rotation of 2.7%, at which very extensive yielding was observed in the two beams just outside of the joint. It appeared that any other loading of the beams beyond this level would produce only limited additional panel zone deformation. A visual inspection of the specimen after seven load cycles showed small cracks forming in the beam flange connection welds. The test was stopped after seven cycles.

The maximum load reached during the final cycle was 495 kN, which was very close to the plastic limit load of the beam, 488 kN. The hysteresis loops of the first, second, third and seventh cycles are shown in Fig. 3. They exhibit the usual stable characteristics associated with steel structures prior to failure due to fracture or instability. There was very substantial strain hardening which occurred almost as soon as the critical region of the panel was yielded.

3.3 Joint B

The specimen was tested with the same load and panel zone deformation increments as Joint A. The removal of the doubler plate reduced greatly the shear resistance of the panel zone and the maximum beam load. Most of the yielding therefore occurred in the panel zone. In fact, the purpose of this test was to demonstrate that the panel zone had adequate ductility and could be subjected to large cyclic distortions without failure.

A total of seven inelastic load cycles were applied, and the range of panel zone rotation was between +4% and -6.2%, the latter was limited by the stroke of the jacks used to load the beams. There was no visible distress in the beam flange welds at these large distortions. The results of the first three cycles as well as the last cycle are shown in Fig. 4. Strain hardening of the panel zone was also very pronounced and the test loads were found to be substantially higher than that calculated by the von Mises criterion.

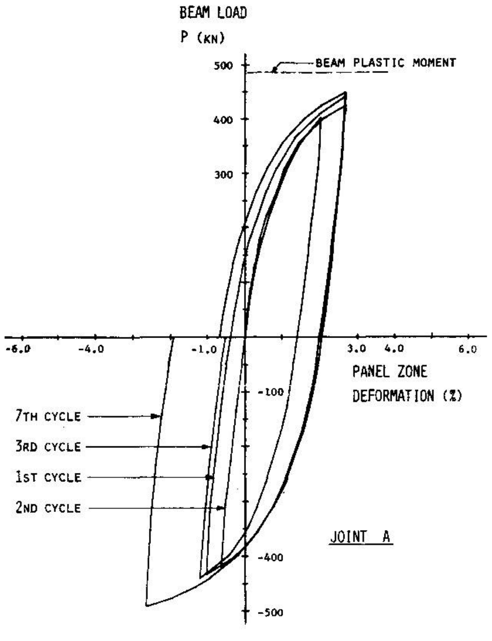


Fig. 3 Load-Deformation Curves of Joint A

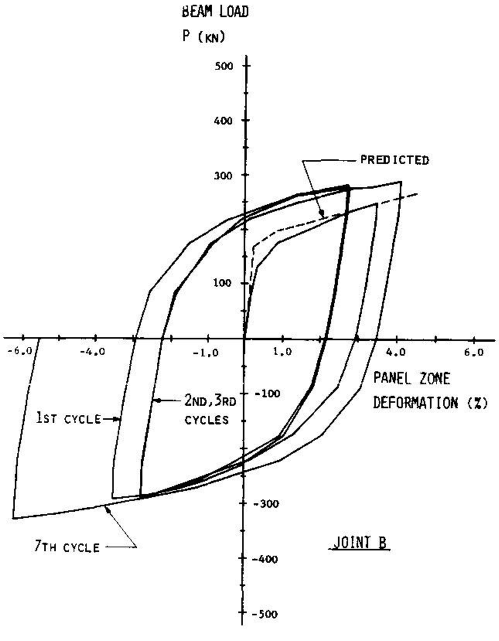


Fig. 4 Load-Deformation Curves of Joint B

3.4 Joint C

The same procedure was again followed in this test. Because earlier studies on joints without continuity plates had indicated significantly less ductility, it was decided for this test to reduce the range of panel zone rotation to about 3.0%. In the first and second load cycles, this joint behaved very much like Joint B, but the removal of the continuity plates apparently had some effects on stiffness. The specimen exhibited a well-defined panel zone for resisting shear. This is illustrated in Fig. 5, which also shows the yield lines in the column flanges opposite to the beam flange welds. The specimen failed at the fourth cycle by a crack through one of the column flanges at the edge of a beam weld.

The results of the test are given in Fig. 6. The decreased slope of the load-deformation curve before fracture indicates that cracks may have developed in the column flange during the previous cycle.

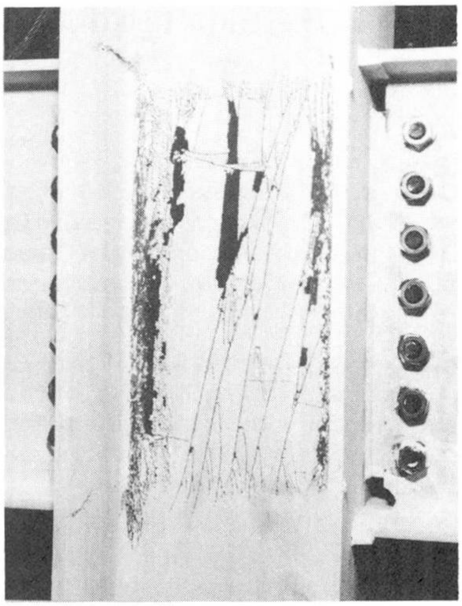


Fig. 5 Panel Zone Yielding of Joint C

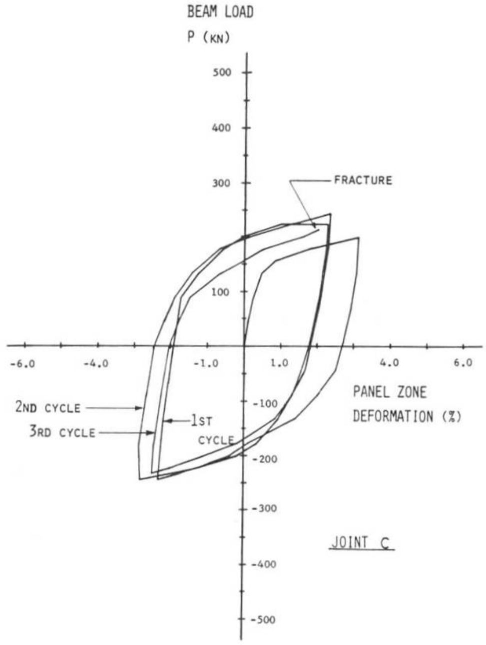


Fig. 6 Load-Deformation Curves of Joint C

3.5 Joint D

A total of 37 load cycles, 24 of which caused inelastic deformation of the panel zone, were applied to the joint. The cycles involved continuously increasing deflections of the load points on the beams, which were used to control the test. The concrete slab cracked in tension very early but continued to provide compressive resistance when the direction of the beam moment was reversed. The specimen failed when cracks developed near the coped holes in the tension flanges of the beams. Such a crack is shown in Fig. 7.

This joint is similar to Joint B in that the panel zone alone was insufficient to resist the shear. Substantial inelastic deformation must occur in the panel zone. In Fig. 8 the total beam load ($P_1 + P_2$) is plotted against the panel

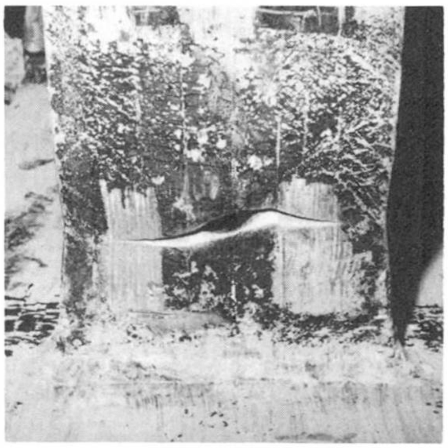


Fig. 7 Fracture of Beam Flange in Joint D

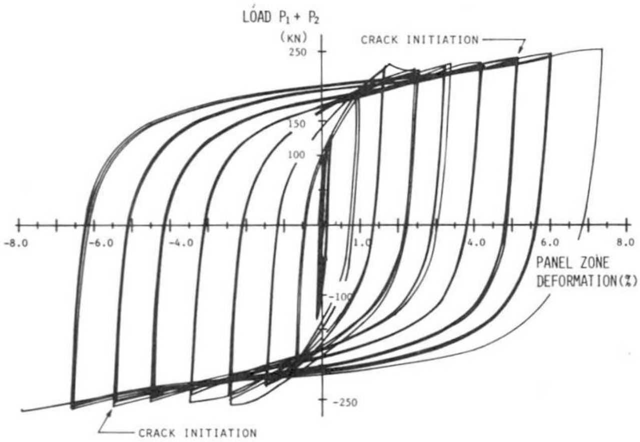


Fig. 8 Load-Deformation Curves of Joint D

zone rotation for all the load cycles. Crack initiation in the beam flange was observed at a panel zone rotation of about 5%, and the maximum rotation achieved was more than 6%.

4. DISCUSSION

Joint A represents the situation in which the designer wishes to utilize both the panel zone rotation and beam yielding for energy absorption. This concept has the advantage of reducing the ductility demand on the beam and its connection to the column flange, thus producing a more balanced design. The panel zone rotation achieved in the test was 2.7%. Based on this value and the theoretical calculations of the inelastic deformation capacity of the beams, a story drift of more than 4.5% has been estimated.

The W24 x 62 beam is unique in that a substantial portion of its plastic moment is contributed by the web. Based on the measured yield stresses of the flange and web of the beams of Joint A, this contribution is found to be 40%. A generally accepted concept of designing connections with fully welded flanges and bolted web is to assume that all the bending moment is resisted by the beam flanges and all the shear resisted by the web. To satisfy this condition, the beam flanges must strain harden sufficiently to make up the difference between the full plastic moment of the section and the plastic moment provided by the flanges. This may become a severe problem for sections with a large portion of the plastic moment provided by the web. However, the test results of Joint A do not seem to indicate this to be particularly serious.

Another feature of Joint A is the use of fillet welds in welding the doubler plate to the column. This procedure, which is less costly, appears to be a satisfactory alternative to full penetration welding.

Joints B and C, both without shear reinforcement, simulate the joints in a frame in which the beams are over-sized for drift control and inelastic action of the panel zone is expected to absorb the energy input. The highly ductile behavior of the panel zone in Joint B indicates the possibility of utilizing shear yielding for energy absorption. The behavior of joints with panel zone yielding can be predicted by the method proposed by H. Krawinkler [3]. In this method, the inelastic deformation of the panel zone is assumed to occur in three stages: shear yielding of the web panel, formation of plastic hinges in the column flanges, and strain hardening of the web panel. This method has been applied to predict the load-deformation relationship for Joint B and the results are shown in Fig. 4. The web panel is fully yielded at a load of 171 kN, but, because of column flange yielding and strain hardening, the maximum load reached in the test was 325 kN, an increase of 90%.

The relatively poor performance of Joint C is a problem of concern and is being carefully examined. A finite element study made on joint geometry has revealed that there is a severe stress concentration in the column flange where the beam flange is attached in the region adjacent to the web when there are no continuity plates. It appears that adequate ductility is very much dependent on having continuity plates of some size in the panel zone.

The results of Joint D test again shown highly ductile behavior of the panel zone. Very substantial strain hardening also occurred, which allow the adjoining beams to yield extensively before fracture of the tension flanges. The envelope or skeleton curves of the hysteresis loops of Fig. 8 are shown in Fig. 9, where the theoretical prediction based on Krawinkler's method is also given. The composite action of the slab makes it difficult to define a proper panel zone height. The results given in Figs. 8 and 9 assume a panel zone height equal to the distance between the continuity plates. The actual height may be larger. The theoretical prediction, which neglects the contribution of the composite slab, is shown to be very conservative.

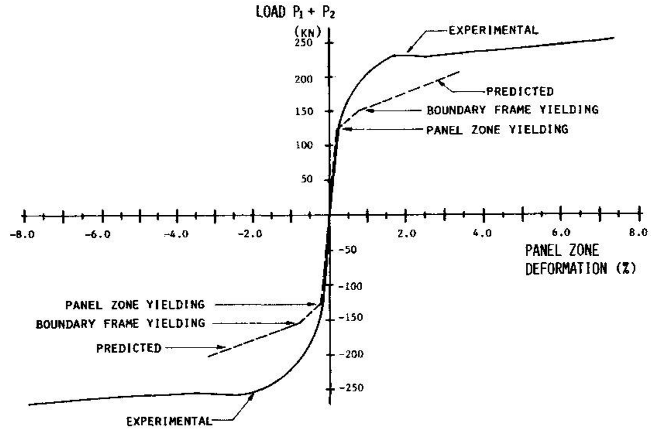


Fig. 9 Skeleton Curves of Joint D

5. CONCLUSIONS

The following conclusions may be drawn from the results presented; they are applicable to joints with dimensions and member sizes comparable to those of the test specimens.

1. The web panel and its boundary elements in a joint with continuity plates can deform inelastically through large shear distortions. A panel zone rotation of 5 to 6% may be achieved with substantial strain hardening.
2. The ductility of joints can be severely imparied when continuity plates are not provided. The joint may fail by cracks through the column flanges adjacent to the beam flange connection welds.
3. For joints designed to develop the plastic moment capacity of the beams, it may be beneficial to allow limited yielding in the panel zone in order to reduce the ductility demand on the beams and the connecting elements.
4. When over-sized beams are used for drift control, shear reinforcement of the column web may not be necessary if sufficient panel zone ductility is available.
5. The panel zone in a composite beam-to-column joint can also behave ductilely and it is possible to achieve an inelastic rotation comparable to that of a non-composite joint.

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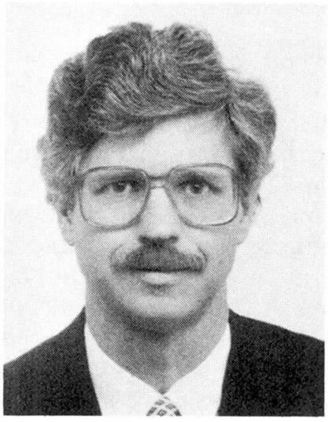
Developments in the Design of Steel Buildings for Earthquake

Progrès dans la conception de bâtiments antisismiques en acier

Neue Entwicklungen in der Konstruktion von erdbebensicheren Stahlbauten

C. Mark SAUNDERS
Vice President
Rutherford & Chekene, Consult. Eng.
San Francisco, CA, USA



C. Mark Saunders holds a B.S. degree from Santa Clara University and an M.S. degree from Stanford University. Over the past 15 years, he has designed numerous structures including steel structures of construction cost up to 80 million dollars (1985). He is Vice President of Rutherford and Chekene Consulting Engineers.

SUMMARY

This paper describes new steel structural systems being used in the United States to resist earthquake forces and some improvements to, and innovative uses of, older systems. Included systems are: braced frames, eccentrically braced frames, steel plate shear walls, and steel plate and concrete composite shear walls.

RÉSUMÉ

Cet article décrit de nouveaux systèmes structuraux en acier, actuellement utilisés aux Etats Unis, qui résistent aux forces provoquées par les séismes, ainsi que quelques améliorations des systèmes traditionnels: cadres contreventés, cadres contreventés excentriquement, murs de refend muni d'une tôle résistante au cisaillement et murs de refend composites en tôle et béton.

ZUSAMMENFASSUNG

Dieser Vortrag beschreibt ein neues erdbebensicheres Stahlbausystem, das in den Vereinigten Staaten angewandt wird und zeigt einige Verbesserungen und Neuerungen älterer Systeme auf. Behandelt werden folgende Systeme: versteifte Rahmen, exzentrisch versteifte Rahmen, Stahlplatten-Schubwände und Schubwände aus dem Verbund von Beton mit Stahlplatten.

1. INTRODUCTION

Design of buildings to resist earthquakes requires design for cyclic loads the magnitude and dynamic characteristics of which are not determinate. This indeterminacy is due to the compounding of uncertainties in the earthquakes themselves with those of the soil and geology and the building characteristics. Certainly, there is, and has been, considerable study undertaken to decrease the level of uncertainty in all of the above factors and this work is valid and continues to improve our knowledge.

For the majority of building designs, it is not practical, nor warranted, to engage in a complete dynamic study in order to produce a design which will provide for competent earthquake performance. Earthquake codes in the United States dating from 1927 have been based on "equivalent static methods" wherein a certain percentage of a building's weight is applied as a static horizontal force distributed over the building height in some fashion. Using these loads, member forces are determined by analysis and members are designed elastically. The earliest of these codes used very simple formulas and relatively low forces. Gradually as our knowledge has improved through research, we have arrived at more and more sophisticated formulas. The present formula, being used in most of the United States (contained in the Uniform Building Code), includes considerations of locality, soil conditions, building period of vibration, framing system type, and importance of the facility. The seismic lateral forces obtained from the code formula are recognized to be considerably lower than those to which the building may in reality be subjected (probably 1/3 to 1/6 of what may occur in a very major quake). It is nonetheless felt that properly executed designs utilizing these forces will produce competent earthquake resistant structures, and for the most part, such has been demonstrated in recent earthquakes.

What is the key to this performance by apparently "underdesigned" structures? It is primarily the post-elastic behavior of the materials and systems. Given materials able to provide large inelastic strains without failure, and systems which preclude instability and brittle connection fracture, large amounts of earthquake input energy can be dissipated by local yielding of the structure, without failure.

Structural steel is, of course, the most outstanding structural material available to meet the requirements of seismic design. The purpose of this paper is to describe some of its newer innovative uses in seismic design of buildings in the United States.

Historically, since our codes were developed, the most commonly used earthquake resisting building sytems for major structures have been moment resisting frames of steel and concrete, concrete shear walls, and steel braced frames for light buildings. Somewhat more recently steel perimeter frames which act as large tubes have been popular for tall buildings of appropriate shape. In recent years several factors have contributed to development and use of new and innovative seismic resisting systems. These factors, all of which ultimately relate to the system economics, include:

- Code imposed limits on building lateral deflections.

- A trend in certain building types to more open and flexible space planning (i.e., more widely spaced columns, movable partitions and higher bays).

- Code imposed higher seismic loads especially for special buildings such as hospitals, public safety structures and large public assembly buildings.

- The extensive seismic upgrading of old, heavy structures of masonry and concrete inspired by government and institutional programs and by tax incentives and preservation requirements for private developments.

The traditional seismic force resisting systems previously listed, while still appropriate and in use for many structures, frequently have liabilities in responding to one or more of the above factors. For example, for large open bay structures, moment frames are frequently uneconomical due to the large members required to limit lateral drift. Concrete shear walls, while excellent for limiting drift in low and midrise structures, have severe architectural liabilities for many structures and, due to high forming costs in the U.S., are frequently uneconomical. Steel braced frames, as currently being designed, while also excellent for limiting drift, have questionable post-elastic performance particularly when used in large heavy structures. Research and design innovations, frequently sponsored by steel industry organizations, have led to the use of new steel systems and to improvements in conventional systems to meet the challenges presented, these include:

- Improved design concepts for concentrically braced frames;

- Steel eccentrically braced frames;

- Steel plate shear walls;

- Steel plate and concrete composite shear walls.

2. DEVELOPMENTS IN CONCENTRICALLY BRACED FRAMES:

Concentrically braced frames (vertical trusses) have been found to be economical systems of lateral bracing for low to moderate height buildings of relatively light weight, and in the United States this has been their predominant use until about the last 10 years.

Use of these frames for larger and heavier structures has become more prevalent largely due to their inherent lateral stiffness to meet the new drift limitations and their economy and convenience as a seismic resisting system in rehabilitation of existing concrete and masonry structures.

It has been recognized for some time that these systems have inherent liabilities in the post-elastic range, since the majority of yielding and therefore energy absorbing capability is concentrated in the brace elements which alternate between tension and compression. The tension yield of the brace results in decreased compression capacity and stiffness of the brace with each successive cycle, leading to continually increasing deflections and possible eventual failure.

The above noted-weakness of this system was previously accounted for in the Uniform Building Code in a rather arbitrary fashion by requiring that members and connections of braced frames be designed for forces 25% larger than those obtained from the code seismic analysis. While perhaps qualitatively correct, quantitatively this increase had no rational basis.

Because of the increased use of braced frames in larger, heavier and more important structures, it was felt that improved design requirements were needed. Recent synthesis and interpretation of research by the Research

Committee of the Structural Engineers Association of Northern California, by an ad hoc committee on Synthesis of Steel Research for Code Development sponsored by the Structural Steel Educational Council, and by the Steel Subcommittee of the Seismology Committee of the Structural Engineers Association of California has led to what is, in my opinion, a more rational approach.

The current approach, which is under study for adoption in the codes is expected to include a reduced capacity for braces based on study of cyclic load test data for compression members. The result is anticipated to be reduction of normally used capacities of long slender braces (length divided by radius of gyration in the range of 120) of 1/2 or more. Shorter braces will suffer lesser reductions. Also included in the revised codes will be required improvements in brace connection details which will greatly decrease the probability of brittle connection failure. These improvements will increase the safety of concentrically braced frame systems and possibly foster even wider use of them.

3. ECCENTRICALLY BRACED FRAMES

The eccentrically braced frame is unquestionably the most popular and intriguing of the new systems, because it combines the stiffness advantages of the braced frame with post elastic performance, comparable in its ability to dissipate energy, to the ductile steel moment frame. An eccentrically braced frame has been defined as "a braced frame in which at least one end of each brace frames only into a beam and in such a way that at least one stable ductile link is formed in each beam". If one thinks of framed lateral force resisting systems as a continuum between the extremes of the moment frame, which depends primarily on bending and shear resistance of the frame elements, to the normal braced frame which depends primarily on the axial strength of diagonal members, the eccentrically braced frame would represent the entire array between the extremes. The degree of reliance of the system on bending and shear, versus its reliance on brace axial forces, is primarily a matter of frame aspect ratios.

Although eccentrically braced frames have been used for years somewhat accidentally, the actual rational development of the system for use in resisting earthquakes has been relatively recent. This development is largely due to the work of Egor Popov at the University of California at Berkeley and his various collaborators.

The system performs, as suggested above, as a hybrid between frame action and braced frame action. The bracing provides excellent stiffness useful in limiting building lateral deformations, while the link beam element is designed as a "fuse" to limit the force in the braces and thus prevent non-ductile type failures such as tension failure of the brace connection or buckling of the brace. The action of the link, particularly when it is designed to yield in shear before it yields in bending, is a particularly effective energy dissipator. A feeling for this energy dissipation can be obtained by examining Fig. 1, which is representative of the type of open, stable hysteresis loops observed by Popov in testing of properly designed shear links.

The design of an eccentrically braced frame system is predicated on the following:

- The link beam should be capable of large inelastic vertical deformations (on the order of 10% of its length) through a number of cycles without buckling or tearing failure.

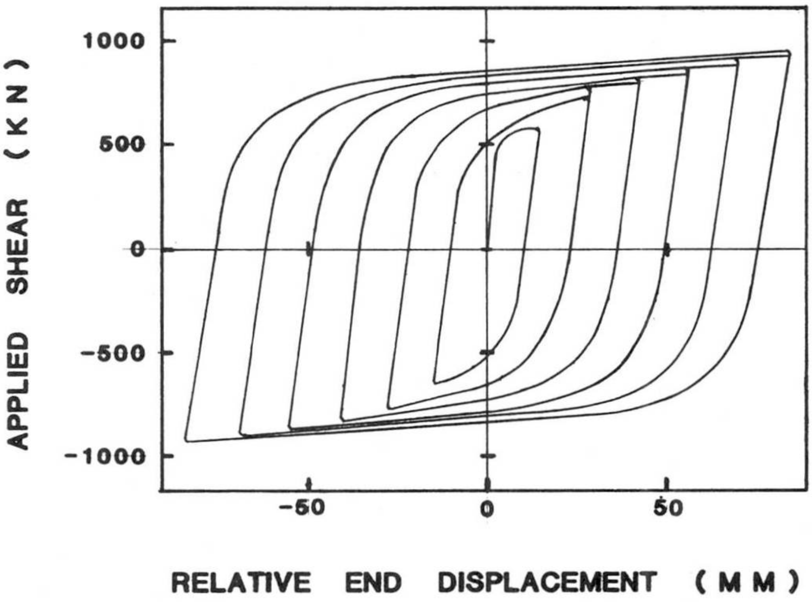


Fig. 1

- The strength of the brace and its connection should exceed by a comfortable margin the yield capacity of the link beam.

- Columns and other elements of the system should be capable of resisting elastically the forces occurring at yield of the link beam.

Once the link beam yields, it acts as a fuse to protect the balance of the system from further increases of loading (except for secondary effects such as strain hardening).

Specifics of the design of this system and the research leading to the design procedure are beyond the scope of this presentation. An extensive list of references on the subject is included at the end of this paper.

As suggested above, this system is highly advantageous when there is need for high ductility and energy absorption coupled with high lateral stiffness.

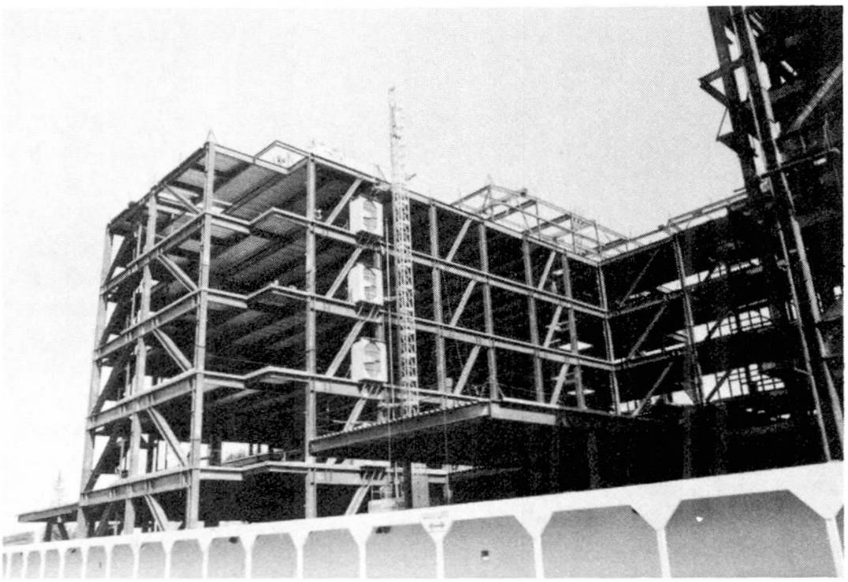


Fig. 2 Eccentrically Braced Frame Hospital

Photographs of a hospital structure employing an eccentrically braced frame system, designed by our office, are included herein. The structure and use are fully described in References [1] and [10].

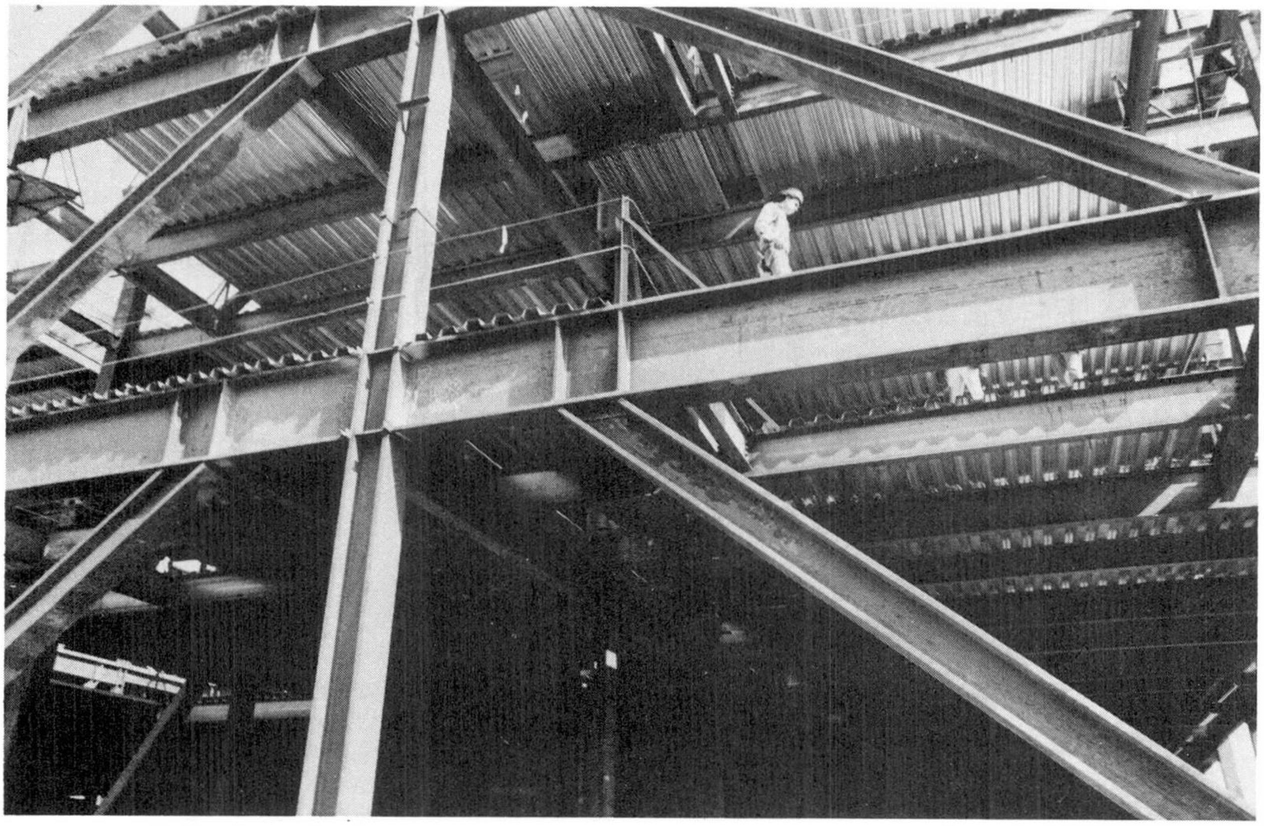


Fig. 3 Eccentric Brace Detail

4. STEEL PLATE SHEAR WALLS

In addition to the use of braced frames to provide seismic bracing for existing heavy and stiff but non-ductile buildings, another novel use of steel has been the steel plate shear wall.

This system has been used in reinforcing an existing hospital building in Charleston, South Carolina where the engineer (URS/John A. Blume and Associates of San Francisco) found that it provided the unique combination of required lateral stiffness and strength with relative ease of erection within a functioning hospital structure.

Each shear wall panel was fabricated in place by field welding a system of plates approximately one meter square and 8 mm in thickness to vertical stiffening ribs made of steel channels approximately 180 mm deep and to horizontal plate stiffeners. The panel size and thickness were determined based on considerations of stiffness and plate buckling and on ease of transportation within the hospital. The panels were attached to existing concrete columns and slabs using drilled-in anchors. A photograph of a typical shear wall panel is included on the next page. Further information on the design of the system can be found in References [2] and [3].

A similar system has also been used in some new structures in both the United States and Japan.

Research on the performance of steel plate shear walls has been reported by Geoffrey Kulak as noted in Ref. [5].

Fig. 4 Steel Plate Shear Wall

5. COMPOSITE STEEL PLATE AND CONCRETE SHEAR WALLS

Another system, which should be mentioned among innovative seismic resistant designs using steel, is the composite steel plate and concrete shear wall system used by H.J. Degenkolb and Associates in designing a 15 story addition for a San Francisco Hospital. This system is fully described in Ref. 4 and will not be repeated in detail herein, except to note the reasons for its use:

- Very high seismic design forces.
- Strict code limitations of lateral deflections (drift).
- Impracticality of moment frames because of member depth, since floor to floor height was limited to match an existing building.
- Extreme thickness required for concrete shear walls (1.2 m at lower levels).
- Difficulty of connections for braced frames under the extreme seismic loadings.

The shear walls consist of steel plates cast into concrete walls. The concrete is held to the steel plates using reinforcing bars through holes in the plates. The concrete is moderately reinforced and is intended to provide lateral stiffness to prevent buckling of the plates and of course will provide considerable dynamic damping to the structure.

ACKNOWLEDGEMENTS

I am grateful to the International Iron and Steel Institute and the American Iron and Steel Institute for their support in the preparation and presentation of this paper. I am also grateful to Joe Baldelli of URS/Blume, Chris Poland of H.J. Degenkolb and Associates and to Prof. Egor Popov.

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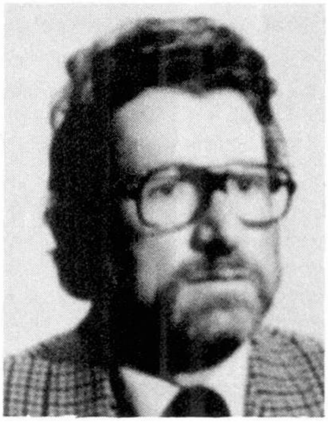
Seismic Resistant System for a Composite Steel-Concrete Building

Système antisismique pour un bâtiment mixte acier-béton

Erdbebensicheres System für ein Gebäude in Stahl-Beton Verbundbauweise

Federico M. MAZZOLANI
Prof. of Struct. Engineering
University of Naples
Naples, Italy



Federico M. Mazzolani, born 1938 in Milan, received his civil engineering degree at the University of Naples in 1961. Author of about 130 scientific papers mainly in the field of stability, plasticity and metal construction. From 1970 to 1979 Chairman of the ECCS Committee «Aluminium Alloy Structures». At present Chairman of the ECCS Committee «Seismic Design».

SUMMARY
This paper briefly describes the structural criteria which have been followed in designing the building for the new Fire Station in Naples. Starting from a previous architectural solution, the structure has been adapted to resist design seismic forces recently introduced in this area. The structure is a composite steel-concrete system in which four floors are hung from a top grid. Special devices have been introduced in order to guarantee a proper seismic risk protection due to the relevant importance of this kind of building.

RÉSUMÉ
Cet article présente brièvement les critères structuraux qui ont été utilisés pour la construction du nouveau bâtiment des pompiers à Naples. Partant du projet architectural, la structure a été adaptée pour résister aux normes sismiques récemment mises en vigueur dans cette région. La structure mixte acier-béton est constituée de quatre planchers suspendus à un treillis supérieur. Des dispositifs spéciaux ont été introduits pour assurer une sécurité suffisante contre les séismes, compte tenu de l'importance de ce type de bâtiment.

ZUSAMMENFASSUNG
Der Vortrag beschreibt kurz die baulichen Kriterien, welche bei der Planung des neuen Feuerwehrgebäudes in Neapel befolgt werden mussten. Ausgehend von einer früheren architektonischen Lösung wurde ein Tragwerk gewählt, welches der seismischen Belastung, die erst vor kurzem in dieser Gegend eingeführt wurde, widersteht. Die Konstruktion besteht aus einem Stahl-Beton Verbund-System, bei dem vier Stockwerke an einem Dachfachwerk aufgehängt sind. Spezielle Vorrichtungen wurden eingeführt, um eine einwandfreie Erdbebensicherheit dieses wichtigen Gebäudes garantieren zu können.

1. INTRODUCTION

A foundamental aspect of an appropriate structural design is to armonize shape and strength of the building. This trend becomes a determinant requirement when the construction belongs to a seismic area and in particular its seismic resistence is of capital importance for civil protection.

Seismic resistant structures are usually designed by introducing in the load combination appropriate enhancement factors which affect the seismic action in order to adapt the degree of seismic protection to the social and economic significance of the relevant building category.

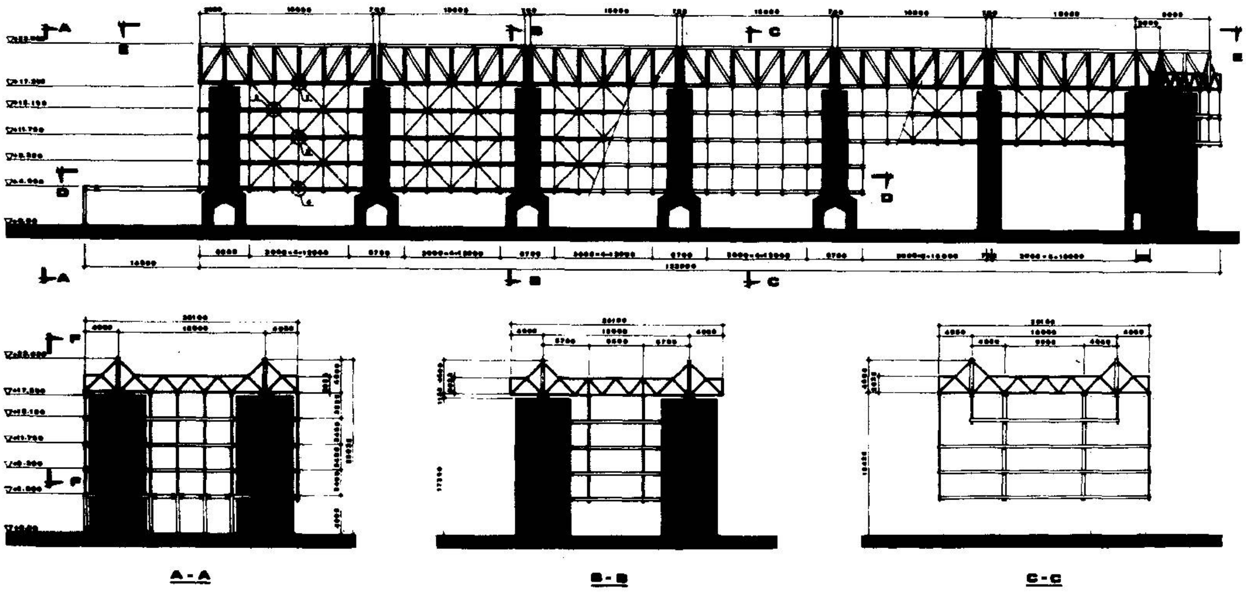


Fig. 1 - General view of the structure.

In the case of important buildings as hospitals, electricity plants, fire stations, etc. the degree of seismic protection is taken into account by means of structural factors. Incidentally in these cases of highest importance the Eurocode "Earthquake" recently suggested a value of 1.4.

Under the design earthquake actions given by codes, structures have to exhibit a given combination of strength, deformability and energy-absorbing capacity. Strength is necessary to withstand the design seismic actions while remaining elastic. Deformation means that structures must be able to safely deform beyond their elastic limit during severe earthquakes and survival is due to their capability to undergo inelastic deformations. Large deformations are a necessary prerequisite for significant energy absorption. The reliability of the solution is reached by providing a suitable combination of these main behavioural aspects.

The structural system used in the new fire station building in Naples (Italy) represents - in our opinion - a significant example which show how a building, which has been previously designed on the main basis of the architectural requirements without considering seismic actions, could be adapted to face the seismic actions with a given seismic risk protection.

2. DESIGN CRITERIA

The structural system adopted for the fire station building belongs to the category of steel-concrete composite structures, where the main vertical bearing elements are the reinforced concrete towers containing stairs and elevators (Fig. 1).

The steel structure, which represents a rigid skeleton, is completely supported by the concrete towers. The floor structures are hanged to a top grid made of longitudinal and transversal reticular girders (Fig. 2).

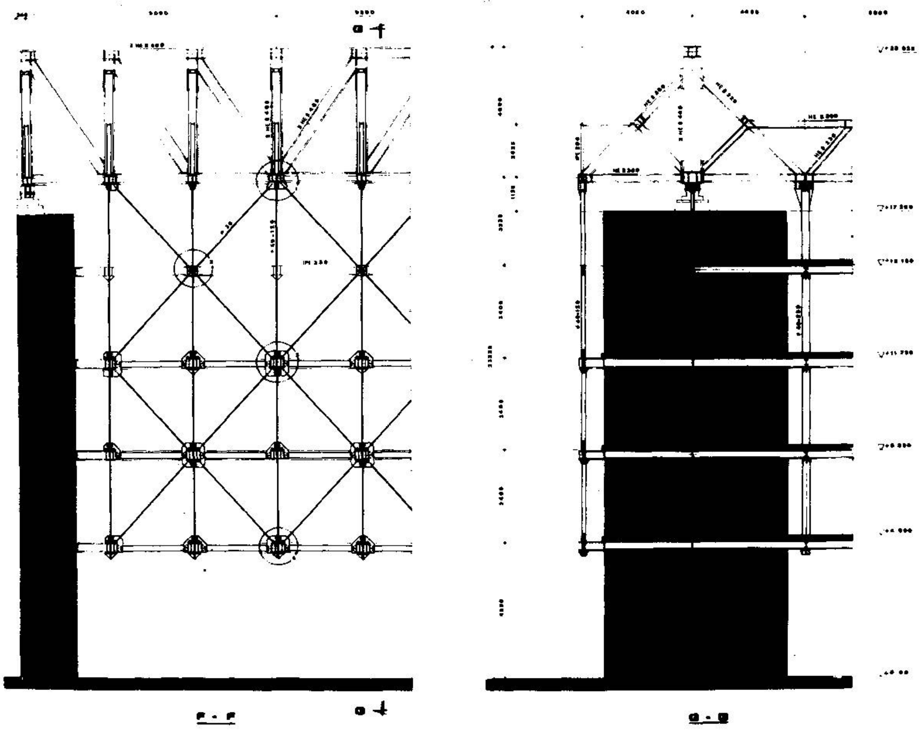


Fig. 2 - Longitudinal and transversal view of the suspended steel skeleton.

This solution was chosen in strict accordance with the architectural requirements which wanted to have a ground floor completely free of structural steel elements.

In the first approach the advantages of this kind of composite system were emphasized, by underlining that:

- reinforced concrete was used for stocky compression elements providing the main vertical and horizontal bearing function;
- steel was used for beams and ties forming the suspended skeleton which resist vertical dead and live loads.

No mention to seismic resistance because at that time Naples was not considered as a seismic area. In the meantime the important earthquake of November 1980 caused the insertion of this town in a new low risk seismic area. As a consequence, it was necessary to adapt the previous design to the new seismic requirements and it was decided to make it by keeping the same architectural solution of hanged construction.

The suspension composite structure was, therefore, interpreted at the light of the new seismic forces and made able to resist them. It was observed that the suspended steel skeleton behaves as a rigid body during the earthquake attack, undergoing vertical and horizontal displacements, transmitted by the isostatic vertical elements.

It seemed, first of all, necessary to fulfil the following requirements:

- the reinforced concrete towars must completely resist the horizontal quakes;
- the steel suspended structure must mainly resist the vertical quakes;
- horizontal and vertical movements must be free under serviciability conditions, but they must allow a proper energy absorption during the earthquake attack.

This last requirement has been satisfied by introducing special devices for seismic risk protection as supports of the suspension system on the top of each tower (see section 5). The dumping effect is obtained by the deformation of rubber layers together with the yielding of appropriate steel elements.

Fig. 3 - Longitudinal reticular girder erected on the top of the concrete towers.

3. STRUCTURAL SYSTEM

The building is composed by a ground floor and four raising flours. Its plan has an extended rectangular shape 26 meters wide, which is longitudinally subdivided following a modulus of 3 meters (Fig. 2).

The reinforced concrete towers, which are coupled in transversal sense, are spaced of 18 meters by forming a square mesh of 18x18 meters (Fig. 1).

The longitudinal reticular girders of the suspension system are simply supported on the top of the towers (Fig. 3). The transversal reticular girders are spaced of 3 meters in correspondence of the vertical members of the longitudinal girders (Fig. 4).

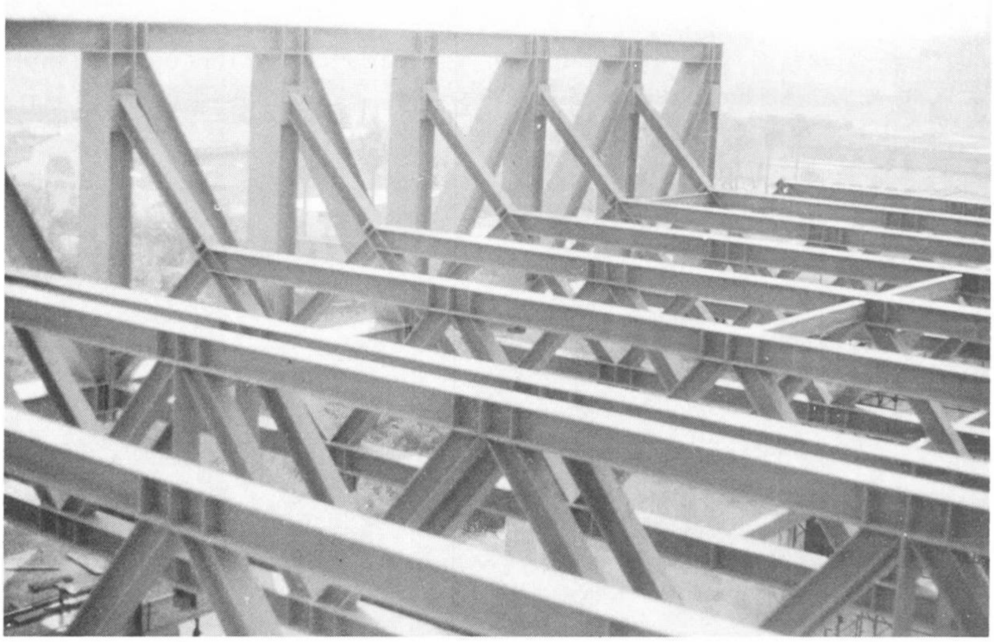


Fig. 4 - Transversal reticular girders of the suspension top system.

The suspension ties, which are hanged to the bottom chords of the reticular girders, support the floor structures at different levels (Fig. 5). Couples of transversal double T beams are suspended at each floor every 3 meters, which is the span of the corrugated sheets filled with concrete. The floor system, in which slabs are shear connected to beams and integrated with horizontal steel bracings, realizes a rigid element against horizontal actions. Vertical bracings are located on the longitudinal perimeter in external position in front of the curtain-walls (Figg. 2 and 6).

4. CONSTRUCTIONAL DETAILS

The principal steel joints have been conceived as full strength connections [1,2]. The butt-welded solution has been adopted for the longitudinal and transversal girders of the suspension system. This led to complete prefabrication of the main girders in the workshop, followed by the transportation of wide (up to 18 meters long) and heavy (up to 33 tons) elements and the erection in their right position at the top level (Fig. 3).

The connection in situ are made by means of end plate and cover plate joints with high strength steel bolts in calibrated holes in order to practically eliminate any slip due to hole-bolt clearance.

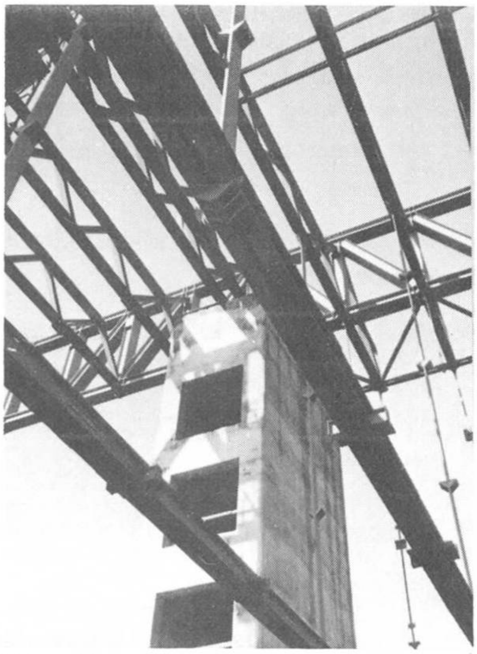


Fig. 5 - Ties and floor beams hanged to the top grid.

5. SEISMIC DEVICES

Technical literature gives different solutions for base isolation systems both for concrete and steel structures [3 to 9]. They are mainly based on the use of rubber together with steel elements plastically working in shear, bending, torsion, etc.

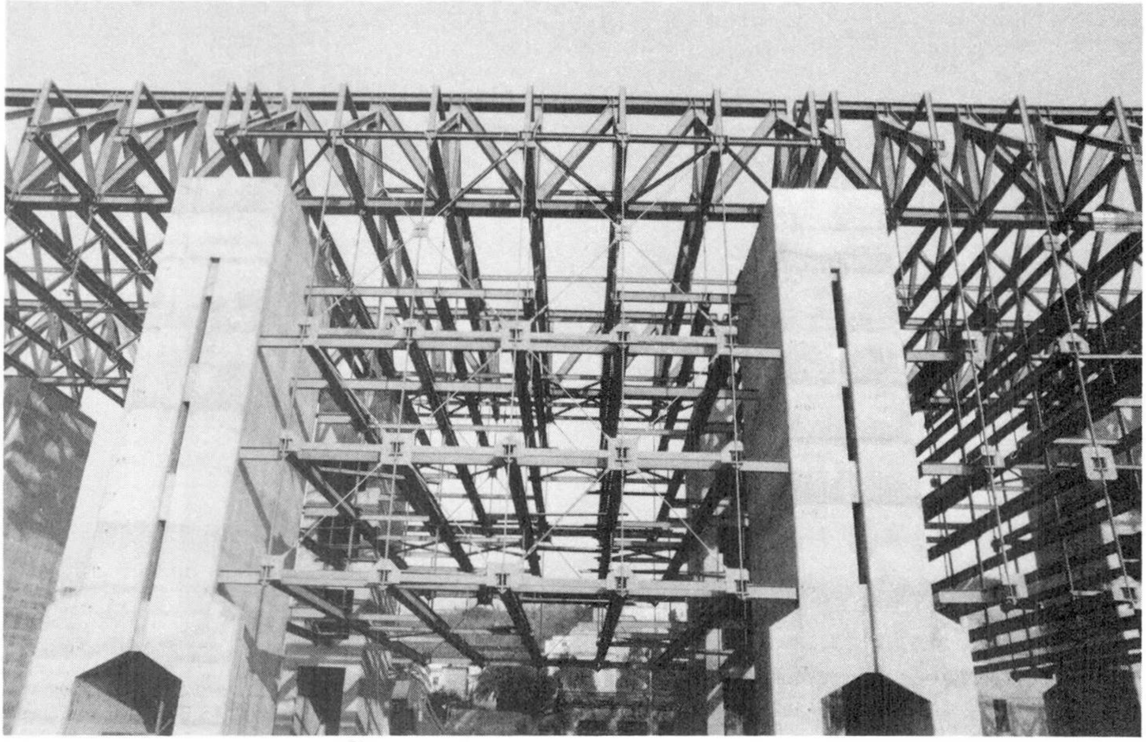


Fig. 6 - Lateral bracings between two towers.

In our case the steel skeleton containing the fire station will undergo horizontal and vertical movements which are transmitted by the concrete towers during the earthquake attack. This skeleton is considered the object to be protected by placing an isolation system at the base of the steel structure where there is the source of the shock.

The supports of the steel skeleton at the top of the towers must play a double function (Fig. 7) :

a) to allow the free movements of structure when it is subjected to the service loads (dead and live loads, wind, temperature...);

b) to damp the horizontal and vertical displacement when it undergoes the seismic attack.

Function a) is given by means of the usual supports which realize fixed or moving hinges.
Function b) is mainly directed to the vertical quakes because they seem to be rather dangerous for a suspended structure. It is obtained by introducing hysteretic dampers in conjunction with flexible layers made of rubber bearing pads (Fig. 8).

The dampers are based on the plastic deformation of steel elements, which absorb energy in two different ways. Due to vertical movements damper components undergo inelastic deformation in tension. Due to horizontal movements damper components inelastically deform in shear and bending.

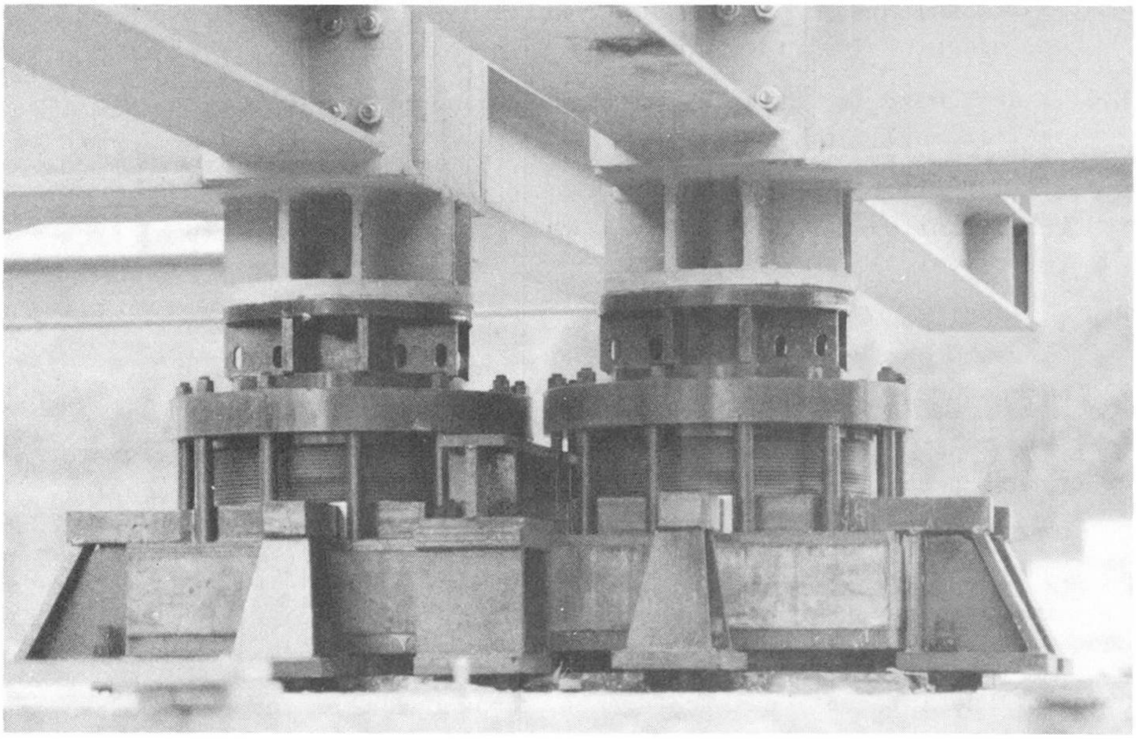
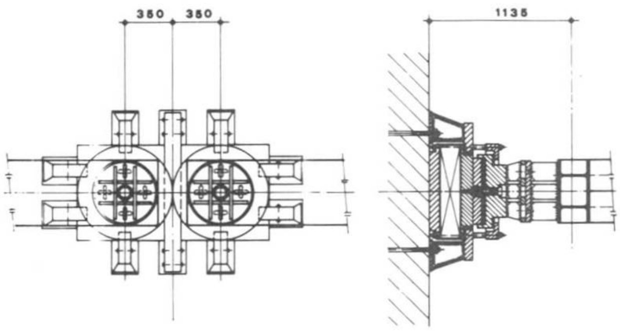


Fig. 7 - Seismic supports of the top grid.

Special devices are also introduced in the floor structure in order to create a flexible connection between the horizontal beams and the vertical wall of concrete towers (Fig. 9).



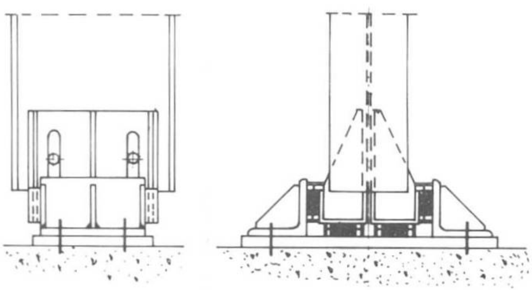


Fig. 8 - Constructional details of the seismic supports.

Fig. 9 - Constructional details of the floor devices.

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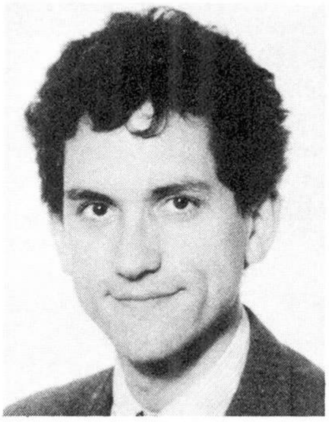
Review of Current and Proposed U.S. Seismic Codes for Steel Structures

Revue des prescriptions sismiques pour les structures en acier aux Etats-Unis

Überblick über die geltenden und vorgeschlagenen amerikanischen Richtlinien für die seismische Bemessung von Stahlkonstruktionen

Guy J.P. NORDENSON
Senior Struct. Eng.
Weidlinger Associates
New York, NY, USA



Guy Nordenson, is a graduate of MIT (BSCE) and the University of California, Berkeley (MS). He has worked in San Francisco, CA and New York, NY, on many earthquake resistant structures throughout the U.S. and in the Middle East. He is an active member of the SEAONC Seismology Committee.

SUMMARY

This paper is a comparative outline of several current and proposed U.S. seismic codes for steel structures. The code requirements for Ductile and Ordinary Moment Resisting Frames, Concentrically Braced Frames, K-Braced Frames and Ductile Eccentrically Braced Frames are discussed.

RÉSUMÉ

Cet article donne un résumé des prescriptions sismiques de construction en acier, courantes et en voie de developement en ce moment aux Etats-Unis. Tout les systèmes de structures en acier permis par ces prescriptions sont discutés.

ZUSAMMENFASSUNG

Dieser Vortrag vergleicht die in den Vereinigten Staaten geltenden sowie die vorgeschlagenen Richtlinien für die seismische Bemessung von Stahlkonstruktionen. Die reglementierten Anforderungen für verschiebliche und biegesteife Rahmen, konzentrisch versteifte Rahmen, K-versteifte Rahmen und verschiebliche, exzentrisch versteifte Rahmen werden erläutert.

1. INTRODUCTION

There are currently in effect almost a dozen design codes for earthquake resistant construction in the United States. These include:

a. The regional model building codes: The BOCA/Basic Building Code (BOCA) issued in Illinois; The National Building Code (NBC) issued in New York; The Standard Building Code (SBC) issued in Alabama; and The Uniform Building Code (UBC) issued in California. In addition there is Standard A58.1, American National Standard Building Code Requirements for Minimum Design Walls in Buildings and Other Structures of the American National Standards Institute (ANSI A58.1). Of these, all but ANSI A58.1 include both loading and material detailing requirements. The latest BOCA, NBC, SBC and the 1972 ANSI A58.1 derive from the "Recommended Lateral Force Requirements and Commentary" of the Seismology Committee of the Structural Engineers Association of California (SEAOC Recommendations) 2nd Edition, 1968. The current 1982 UBC is based on the 1980 edition of the SEAOC Recommendations, and the 1982 ANSI A58.1 is part 1975 SEAOC Recommendations, part ATC 3-06 (see below).
b. The SEAOC Recommendations are currently being revised. The remarks that follow are based on the 15 November 1984 Draft (SEAOC 11:84 Draft) of this document. The draft is still very much in discussion.
c. Codes developped for agencies and services of the Federal U.S. Government include: the General Service Administration which contracts most federal office buildings: the Veterans Administration which mostly builds hospitals the Departments of the Army, the Navy and the Air Force which publish the Technical Manual - Seismic Design for Buildings (1982 Tri-Services Manual).
d. Finally the Applied Technology Council, a non-profit research subsidiary established in 1971 by SEAOC, published the "Tentative Provisions for the Development of Seismic Regulations for Buildings" (ATC 3-06) in 1978. This project was funded by the U.S. National Science Foundation and National Bureau of Standards and undertaken by a group of 85 of the top earthquake engineering professionals and academicians.

The following is a comparative review of the design requirements for steel structures contained in the 1978 ATC 3-06, the 1982 UBC, Tri-Services Manual and ANSI A58.1 and the SEAOC 11:84 Draft only. These five codes best represent the range of consensus among current U.S. seismic codes. The structural systems discussed are Ductile (or Special) and Ordinary Moment Resistant Frames, Concentrically and K-Braced Frames and Ductile Eccentrically Braced Frames. The emphasis will be on two areas where a consensus is still not apparent and where additional "mission oriented" research would be helpful: Ordinary Moment Frames, or in general systems for zones of moderate seismicity, and Concentrically and especially K-Braced Frames.

2. DUCTILE MOMENT RESISTING FRAMES

Among earthquake resistant steel structural systems, Ductile Moment Resisting Frames (DMF) have received the greatest attention in recent research (Popov 1983). The detailing requirements described below have evolved mostly from the results of this research, rather than the evidence of earthquake damage. Indeed few multistory steel DMF's built in accordance to modern U.S. seismic codes have yet to be tested in severe earthquakes. The following requirements are all intended to help insure stable cyclic energy dissipating capacity for large ductility demands.

2.1 Column Strength Requirements

The SEAOC Recommendations (1975 on) suggests in its commentary that DMF's be designed for a "strong column-weak beam" mechanism. It is further suggested, based on the test results of Popov (1975) that the column axial load ration P/P_y be kept below 0.5. ATC 3-06 modified this to require that P be limited to less than 0.6 P_y. Neither the 1982 UBC nor the Tri-Services Manual address this issue. The SEAOC 11:84 Draft includes general provisions for column strength applicable to all steel systems: (a) the axial compressive stress due to gravity loads plus 3 times the modified elastic reponse spectrum given in the code (i.e. $3R_y/8$) must not exceed 1.7 times the allowable stress (F_a) unless that axial load is somehow limited by the mechanism; and (b) the K value used to calculate the effective length of the column can be taken as 1.0 if the P-delta due to the lateral loads is considered explicitly. ATC 3-06 included a similar provision for the K value. The strong column-weak beam principle is quantified in the SEAOC 11:84 Draft for the first time: the sum of column plastic moments (calculated by $Z_c(F_{yc}-F_a)$) is required to exceed the sum of girder plastic moments unless the column axial stress is less than 0.4 F_y.

2.2 Column Splice Requirement

ATC 3-06 limited the use of partial penetration welds in column splices by requiring that they be able to withstand stresses due to either: 1.25 times the full joint plastic moment at both ends; the full joint plastic moment at one end combined with half the plastic moment at the other end; or the tension imposed by half the gravity load minus the seismic axial load. The SEAOC 11:84 Draft currently includes a requirement that partial penetration welds be sized for 150 percent of the tension due to $3R_w/8$ (i.e. 3 times the design spectrum values) times the seismic axial load in conjunction with the gravity load, unless that seismic axial load is limited by the mechanism. The minimum would be to develop 50% of the column flange area.

2.3 Joint Panel Zone Shear Design

The 1982 UBC and Tri Services Manual reference the AISC Code which in its plastic design section limits the panel zone shear stress to 0.55 F_y. Though neither code specifies it, the practice (suggested in the Commentary to the SEAOC Recommendations) has been to check the panel zone for shears due to the development of girder plastic moments. This has often led to the necessity of adding costly column web doubler plates. ATC 3-06 both codified the requirement and provided possible relief by allowing an exception for panel zones capable of resisting stresses resulting from twice the design level story drifts. The SEAOC 11:84 Draft is not yet resolved on this issue. One proposal is to limit the design level (working) shear stresses to 0.4 F_y. This would lead to reductions in doubler plate requirements, but could also result in joints whose strength is governed by that of the panel zone, which can be as little as half the girder strength. Tests (Bertero et.al. 1972) have indicated that such mechanisms can lead to local kinks at the beam/column flange intersection which may cause cracking and fracture at the welds. Krawinkler (1978, 1985) has proposed instead an equation which accounts for the strengthening effect of the column flanges (up to 20-30% for the most common member sizes) in evaluating the panel zone capacity. This could be used in conjunction with either the full plastic girder moments or twice the code level seismic moments, as suggested in ATC 3-06.

2.4 Beam Column Connections

Both the 1982 UBC and Tri-Services Manual require the connection to develop the girder flexural strength or provide "adequate rotation capacity". Where the specified ultimate strength of the steel is less than 1.5 times the yield strength bolted flange connections are prohibited. ATC 3-06 includes only the former requirement. The SEAOC 11:84 Draft is also not yet resolved on this matter. There is concern that where the plastic modulus of the girder flanges alone (Z_{fl}) is less than 70% of the full plastic modulus, the typical butt welded flange and bolted web connection is insufficient (at $Z_{fl} + 0.7Z_b$ the flange stress would be $1.43F_y$). On the other hand, research work (Popov 1983) indicates that such connections perform quite well in cyclic load tests.

3. ORDINARY MOMENT FRAMES

Ordinary Moment Frames (OMF), a term coined by ATC 3-06, are non-ductile or semi-ductile moment frames intended for use in regions of moderate to low seismicity. UBC 1982 had allowed moment frames designed in accordance with the AISC Code to be used in Zones 1 and 2 (of a possible 4) with the same structural system factor (K) as Ductile Moment Frames (DMF). This is by far the least conservative of the requirements under review (see Table 1). ATC 3-06 prescribes force levels 1.78 times the force levels for DMF's for OMF's and allows their use in regions of moderate seismicity (Seismic Performance Categories A and B) with no height limit and below 160 ft. or 100 ft. for normal and essential facility in zones of high seismicity (Categories C and D). The detailing need only satisfy the normal requirements of the AISC Code, with no special provisions for ductility.

The Tri-Services Manual provides for 3 types of moment frames: Type A which corresponds to the DMF; Type B frames which are allowed up to a height limit of 160 ft. in moderate seismic zones and 80 ft. in high seismic zones and which must be sized for twice the seismic moment plus the gravity moment ($2M_e + M_d + M_l$): and Type C frames which are allowed up to 80 ft. in zones of low seismicity (Zone 1) and can be designed by the AISC Code. Girders in Type B frames would therefore have roughly 2.6 times the strength of those in OMF's designed by UBC 1982 (assuming M_e = 75% of design moment).

The 1982 edition of ANSI A58.1 proposes that OMF's be designed for 1.5 times the forces used for DMF's (i.e. K=1.0) and that the beam/column connection develop the joint capacity. The SEAOC 11:84 Draft specifies OMF force levels 2-2.4 times those for DMF's. Furthermore they are prohibited in zones of high seismicity (Zone 3 and 4) unless they can sustain loads 3 times the design response spectrum values, or 4.5 times the force levels for DMF's ($3R_w/8$). Otherwise the Draft as of yet provides no detailing requirements.

Clearly there is wide disagreement among codes for OMF's. Prescribed force levels in moderate seismic zones are either 1.0, 1.5, 1.78, 2.4 or 2.6 times those for DMF's. In zones of high seismicity, the ratio may reach as high as 4.5. The problem is that there is at present little research or earthquake reconnaissance data to permit a determination of the cyclic inelastic response of the typical bolted flange moment connections used in areas outside zones of high seismicity. Popov et al. (1969, 1970 cited in Popov 1983) in a series of tests on W8 x 20 beams found that though the butt welded flange/bolted web connection showed greater cyclic energy dissipating capacity than any other type of connection, the bolted or welded flange plate connections did withstand substantial inelastic rotations prior to crack formation at the end weld or outermost bolt line. Interestingly though, connections to the weak axis of columns failed by cracking in a manner similar to the flange plate connections. Such connections are common in DMF's. In any case the hysteresis curves for bolted flange plate connections are "pinched" or S-shaped due to bolt slippage, not unlike those for reinforced concrete DMF's (which have structural system factors no more than 1.2 (SEAOC 11:84 Draft) above those for steel DMF's). It would seem therefore that some inelastic response could be safely permitted in OMF's, suggesting that the requirements proposed in ATC 3-06 and the Tri-Services Manual are in the correct range.

Since seismic code provisions are increasingly being adopted or considered in areas of moderate seismicity (e.g. Boston, Memphis and perhaps New York) a test program to evaluate the cyclic inelastic response of currently common or "vernacular" steel details in those areas would be quite useful.

4. CONCENTRICALLY BRACED FRAMES

The UBC 1982, Tri-Services Manual, and ANSI A58.1-1982 both require that the brace member of Concentrically Braced Frames (CBF) be sized for 1.25 times the axial force resulting from the design lateral seismic load, and that the connections either develop the member or 1.88 times the design axial force (i.e. 1.25 times that force, without the usual one third stress increase). This means that the compression brace will buckle at roughly 1.6 times the design seismic force levels (1.25 x 1.7/1.33). It is essential then that CBF's be detailed to insure good energy dissipating capacity. This is difficult since the hysteresis curves for steel struts under cyclic axial load reversal are pinched and show a significant deterioration in compression capacity (Black et.al. 1980). In an effort to avoid very slender braces, which have performed poorly in past earthquakes, ATC 3-06 requires that the axial compression capacity of a brace be greater than 50% of the tensile strength. For A36 steel this corresponds to a Kl/r limit of 115. The CBF's reserve strength (in X-braces for instance) is thereby doubled as well.

The SEAOC 11:84 Draft requirements for CBF's represent a shift in emphasis to increasing ductility rather than strength. Briefly the provisions include:

a. A limit of $720/(F_y)^{1/2}$ for the brace slenderness ratio L/r (the CBF reserve strength is therefore 2.14 times the design axial load for A36 Steel).
b. A cyclic reduction factor of $1/[(1 + Kl/r)/C_c]$ to be applied to the allowable compressive stress. This provision is still under discussion.
c. A requirement that the brace end connection either develop its tensile strength or 3 times the design response spectrum values ($3R_w/8$).
d. A requirement that in any plane of braced frames an equal amount of compression and tension braces be provided for either loading direction.

Altogether these and other provisions are intended for CBF systems (e.g. X-bracing) which permit the tensile yielding of the brace. Though pinched, the resulting experimental hysteresis curves show stable cyclic inelastic response with no strength deterioration and some energy dissipation. Furthermore, since the brace is sized for compression, the maximum resistance obtained in each half cycle is at least double the design axial force. The important point is that the mechanism assumed (cyclic inelastic buckling and tensile yielding of the brace) is more explicitly manifest in the SEAOC 11:84 code provisions leading a designer to consider how the other elements (beams, columns, connections) must be sized to

Comparison of Lateral Force and Drift Requirements of Several Seismic Codes — Table 1

	UBC 1982	ATC 3-06 [2]	Tri-Services *(ANSI A58.1-82 similar.)	SEAOC 11:84 Draft
Modified Response Spectrum Equation (S = soils factor, max. 1.5; Z, A_a and A_v are zone factors)	$ZS/15T^{1/2}$ (0.14/Z max.)	$1.2A_vS/T^{2/3}$ ($2.5A_a$ or $2A_a$ if S=1.5, max.)	$ZS/15T^{1/2}$ (0.14/Z max.)	$0.5ZS/T^{2/3}$ (1.1/Z or 0.9/Z if S=1.5, max.)
Base Shear Equation	V = ZICSKW	$V = 1.2A_vSW/RT^{2/3}$	V = ZICSKW	$V = ZICW/R_w$
Steel Moment Resisting Frames				
Approx. Period Estimate	0.10 x No. of stories	$0.035h_n^{3/4}$	0.10N	$0.035h_n^{3/4}$
Structural Sys. Factor	K	R	K	R_w
Ductile Mom. Frms.	0.67	8	0.67*	12
Ordinary Mom. Frms.				
High Seismicity Zone.	not incl.	not incl.	1.0 (1.5)[1]* (Type B, h_n less than 48m)	5 or 6 (2-2.4) (h_n less than 48m)
Moderate Seismicity Zone.	0.67 (Zone 1&2)	not incl.	1.0 (1.5)* (Type C to 24m in Zone 1, B to 48m in Zone 1&2)	5 or 6 (2-2.4) (no height limit)
Elastic Drift Limit	0.005K	$0.015/C_d$x1.4	0.005K	$0.04/R_w$
Ductile Mom. Frms.	0.0033	0.0019	0.0033	0.0033
Ordinary Mom. Frms.	0.0033 (1)[1] (Zone 1&2)	0.0027 (1.4)	0.005 (1.5)	0.0067 - 0.008 (2-2.4)
Steel Braced Frames				
Approx Period Estimate	$0.05h_n/D^{1/2}$	$0.05h_n/L^{1/2}$	$0.05h_n/D^{1/2}$	$0.05h_n/D^{1/2}$
Structural System Factor	K	R	K	R_w
Concentrically Br. Fr.				
Brg. Wall Sys.	1.33 (2)[1]	4 (2)	1.33 (2)	4 (3)
Bldg. Fr. Sys.	1.0 (1.5)	5 (1.6)	1.0 (1.5)	6 (2)
Dual Sys.				
w/ DMF	0.8 (1.2)	6 (1.3)	0.8 (1.2)	8 (1.5)
w/ OMF	0.8 (Zone 1&2)	not incl.	0.8 (Type B, Zone 1&2)	not incl.
Ductile Eccentrically Br. Fr.				
Bldg. Fr. Sys.	not incl.	not incl.	not incl.	10 (1.2)
Dual Sys. w/ DMF	"	"	"	12 (1)
Elastic Drift Limit	0.005K	$0.015/C_d$x1.4	0.005K	$0.04/R_w$
Concentrically Br. Fr.				
Brg. Wall Sys.	0.0067 (2)	0.0031 (1.6)	0.0067 (2)	0.01 (3)
Bldg. Fr. Sys.	0.005 (1.5)	0.0024 (1.3)	0.005 (1.5)	0.0067 (2)
Dual Sys.				
w/ DMF	0.004 (1.2)	0.0021 (1.1)	0.004 (1.2)	0.005 (1.5)
w/ OMF	0.004 (1.2) (Zone 1&2)	not incl.	0.004 (1.2) (see above)	not incl.
Ductile Eccentrically Br. Fr.				
Bldg. Fr. Sys.	not incl.	not incl.	not incl.	0.004 (1.2)
Dual Sys. w/ DMF	"	"	"	0.0033 (1)

1 Numbers in parenthesis indicate the ratio of the particular value to that for Ductile Moment Resisting Frames.

2 ATC 3-06 is an ultimate design (or Strength design) code. All the others are allowable stress design codes.

realize the mechanism.

Finally, one may note from Table 1 that the SEAOC 11:84 Draft Structural Systems Factors assigned to CBF's are substantially higher than those in the other codes. This reflects the draft state of this document. The factors have yet to adjust to the improvements in the material sections of the draft code.

5. K-BRACED FRAMES

K-Braced Frames (KBF) differ from X-braced or other CBF's in two respects.

a. Once the compression brace buckles, the strength of the frame is dependent on the beam strength only. The brace will not therefore develop its tensile strength.
b. With repeated load reversals, the buckling capacity of a slender (Kl/r over 80) brace can deteriorate to less than 50% of its original value (Jain et.al. 1979).

Together these observations indicate that the strength of KBF's, after several cycles of loading may be only 25% of what a CBF designed per ATC 3-06 or the SEAOC 11:84 Draft would exhibit. Since there is virtually no data on the performance of KBF's in past earthquakes and very little experimental results it is not clear how severe the problem is.

In the mean time, it has been suggested (Nordenson 1984) that one might either limit the brace Kl/r to around 40, lessening the reduction in buckling capacity, or perhaps view KBF's as two phased systems: a CBF up to buckling and an Eccentrically Braced Frame with flexural links thereafter.

6. DUCTILE ECCENTRICALLY BRACED FRAMES

Ductile Eccentrically Braced Frames (DEBF) are gaining favor as ductile alternates to CBF's. These frames are both quite stiff and capable of large ductility and energy dissipating capacity. The inelastic action is localised in a shear or flexural yielding link and the balance of the frame is designed to develop that mechanism (Kasai et.al. 1984). The SEAOC 11:84 Draft is the first seismic code to include DEBF's. The provisions include the following requirements:

a. the compression strength of the brace should exceed 1.5 times the axial load corresponding to the link yield mechanism.
b. the columns should remain elastic for the given mechanism calculated for 125% of the material yield strength.
c. the maximum link end rotation should not exceed 0.06 rad. at $3P_w/8$ times the design drift (= 3.75 to 4.5 depending on the system).
d. connections and elements must satisfy the DMF requirements.
e. brace to beam connections must develop the brace capacity.
f. links should be laterally braced at their ends and have sufficient web stiffeners ty transfer the brace force and insure adequate ductility for cyclic inelastic load reversals.

7. CONCLUSION

A fair degree of consensus exists regarding the design of DMF's and DEBF's, though there is still debate on certain provisions in the SEAOC 11:84 Draft. The requirements for CBF's are still in development and it is not yet clear what the recommendations will be. Still it seems that the direction is toward an increase in dependable ductility through detailing which could result in a slight reduction in required elastic strength.

OMF's and especially KBF's have yet to be seriously considered partly because there is little data or test results available to substantiate code provsions and partly because, in the case of OMF's there is little interest since the codes are generally geared for zones of high seismicity. The problems associated with KBF's, if experiments confirm the weaknesses hypothesized, could be serious, and since these structures are often used, should receive more attention.

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