Zeitschrift: IABSE proceedings = Mémoires AIPC = IVBH Abhandlungen

Band: 11 (1987)

Heft: P-111: Dynamic analysis of steel structures with regard to progressive collapse

Artikel: Dynamic analysis of steel structures with regard to progressive collapse

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DOI: https://doi.org/10.5169/seals-40371

Dynamic Analysis of Steel Structures with Regard to Progressive Collapse

Calcul dynamique de structures métalliques relatif à un effondrement progessif

Dynamische Berechnung von Stahlbauten in bezug auf progressiven Einsturz

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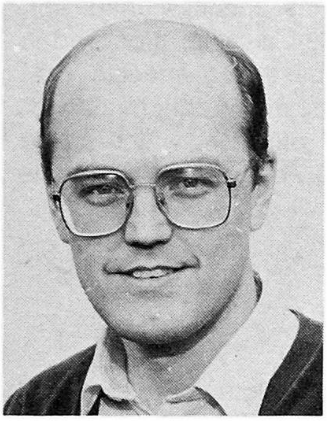

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SUMMARY

The present study deals with dynamic analysis of steel structures with regard to abnormal loading and progressive collapse. Methods of determining damage tolerance of structures are briefly reviewed. The basic effects in steel structures during the stage of dynamic transition between the original and the alternate load-carrying systems of the structure are discussed. Fundamental models of damaged steel-framed structures are proposed in order to analyse and evaluate the influence of the dynamic effects on the design criteria.

RÉSUMÉ

L'étude traite du calcul dynamique de structures métalliques soumises à des charges exceptionnelles et à un effondrement progressif. Des méthodes sont présentées pour déterminer les dommages tolérables de ces structures. Les effets dynamiques fondamentaux sont étudiés durant la phase transitoire entre le système porteur original et modifié. Des modèles sont proposés afin de calculer et d'évaluer l'influence des effets dynamiques sur les critères de dimensionnement des structures métalliques endommagées.

ZUSAMMENFASSUNG

Im vorliegenden Aufsatz wird die dynamische Behandlung des Problems von aussergewöhnlichen Einwirkungen und progressivem Einsturz mit Anwendung auf Stahlbauten behandelt. Es werden Methoden zur Ermittlung der Schadenstoleranz vorgestellt. Die Einflüsse in Stahlbauten, die während des dynamischen Übergangs vom ursprünglichen zum alternativen Tragsystem auftreten, werden diskutiert. Modelle von beschädigten Stahlskelettbauten zur Berechnung und Auswertung dieser Einflüsse auf die Entwurfskriterien werden vorgeschlagen.

1. INTRODUCTION

The susceptibility of structures subjected to abnormal loads and to progressive collapse depends on the structural material, structural system and method of construction used. Large concrete panel and masonry structures are usually considered as being particularly susceptible to progressive failure. However, collapses of many steel and timber structures have also been observed /1,2,3,7/. It is therefore generally accepted that the problem of progressive collapse is of a general nature and should be considered in the design of all types of buildings /4/. Most research work conducted to date in this area has been devoted to large concrete panel structures. In this paper, the progressive collapse of steel-framed structures are studied. At the time when the research work reported in this paper commenced /5,14-16/, no studies on steel structures subjected to abnormal loads and loss of load-bearing elements had been reported /3,7/. However, recently an analysis of the progressive collapse resistance of steel structures has been presented /6/.

In recognition of the problem of progressive collapse, principles for design against progressive failure have been incorporated in the codes of many European countries and in Canada /8/. These principles have recently been incorporated in the standard ISO 2394 and are proposed in the code Eurocode No 1 /9,10/. These design principles are usually based on specific local resistance or alternate load path under static loading conditions. However, many abnormal loads such as explosions and vehicle impacts are phenomena of very short duration, which cause dynamic effects in the structure. In this paper, the dynamic effects of the abnormal loads on the alternate load path in steel structures are discussed /17,18/.

2. BASIC DESIGN PRINCIPLES

The problem of progressive collapse should be recognized as an important element in a general design approach. Abnormal loading and progressive collapse concepts should be integrated into general principles on quality assurance and reliability based design approaches for structures, cf. /11/. Abnormal and accidental loads are highly random in nature. Vehicular collisions and gas and bomb explosions are the major hazards for multistorey residential buildings /12/. The overall failure probability of common types of construction due to such events may in some instances exceed the failure probability due to normal design loads /13/. This indicates the need for including progressive collapse resistance in the design of building structures.

Progressive collapse resistant design implies either reducing the risk of initial damage or preventing a chain reaction of failures following a primary damage. Resistance against progressive collapse can be provided in building structures by either indirect or direct design related to the structural integrity and/or some kind of reserve strength or damage tolerance. The fundamental measures to assure structural integrity and damage tolerance of buildings are to design the structure having: (i) Excess strength; (ii) Redundancy; (iii) Large deformation capacity /15/. These three provisions can separately or in combination increase the damage endurance. By excess strength is meant the capacity of the structure to carry overloads, i.e. to carry the additional loads imposed on the structure subsequent to the occurrence of primary damage. The redundancy implies continuous, unified and anchored structural components. Due to this redundancy the forces originating from a local damage are distributed to a large number of elements in the remaining structure. Thus, the demand for excess strength of the single component is decreased. The deformation capacity refers to the ductility properties of structural members and joints. The deformation capacity reduces the demand for excess strength for a given energy-absorbtion capacity. New load-bearing systems such as catenary action are made possible in a redundant structure when sufficient deformation capacity exists. Indirect design refers to ensuring progressive collapse resistance by specifying a minimum level of those three parameters, i.e. strength, continuity and ductility.

Direct design refers to explicit evaluation of the progressive collapse resistance and the ability of a structure to absorb the local damage. The two basic means of direct design are the specific local resistance method and the alternate load path method /12/. The specific local resistance method implies designing the load-bearing structural elements and joints so that they can resist the abnormal load. The alternate load path method, however, permits local damage and loss of load-carrying capacity of a structural member to occur but provides alternate paths to bridge over the area of primary damage. The design criteria for abnormal load resistance and damage tolerance have been given in the same format as for normal design situations by using the method of partial coefficients. Ellingwood and Leyendecker have evaluated the load factors for the load effects from dead (Q_d), live (Q_l), wind (Q_w), and abnormal (Q_a) loads and the probability of occurrence of wind and/or abnormal loads for the two direct design approaches /12/. The factored resistance (R/γ) and the factored load design equation for the specific local resistance method is

$$\frac{R}{\gamma} \geq 1{,}0\ Q_d + 0{,}4\ Q_l + 1{,}3\ Q_a \tag{1}$$

and the corresponding design equation for the alternate load path method is

$$\frac{R}{\gamma} \geq 1{,}0\ Q_d + 0{,}45\ Q_l + 0{,}2\ Q_w \tag{2}$$

The alternate path and damage tolerance method for controlling progressvie collapse is recommended over the specific local resistance approach /12/. This paper deals only with the method of alternate load path. (As a comparison, it can be mentioned that, according to Eurocode No 1, the factored load design equation for the specific local resistance method is given by (1,0 or 1,35)Q_d + 0,4 Q_l + 1,0 Q_a + 0,2 Q_w for typical dwellings /10/.)

The alternate path approach described above is based on static analyses of the behaviour of the structure before and after the local damage has occurred, and no attention is paid to the transition between these two states. Since the most frequent abnormal loads are pressure loads from gas and bomb explosions and impact loads from vehicle collisions, the load-bearing elements will be removed rapidly and thereby cause dynamic effects in the structure. Thus, in many cases a stage of dynamic transition exists between the original state and the damaged state, see Fig. 1 /15/. A structure may function properly in the original and the damaged states, but may not do so during the stage of dynamic transition. In such a case collapse may occur before the alternate load-carrying system in the damaged structure has come into action. The basic dynamic conditions during the stage of dynamic transition will be discussed and the significance of the dynamic transition stage for an actual practical case will be illustrated in the next section.

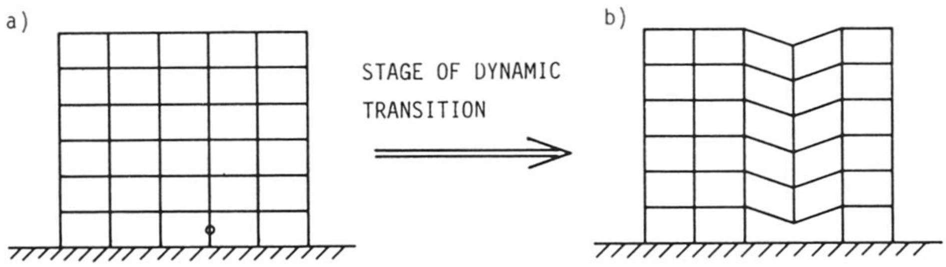

Fig. 1 Stage of dynamic transition between the original (a) and the alternate (b) load-bearing system

3. BASIC DYNAMIC EFFECTS

In this section some basic dynamic conditions during the stage of dynamic transition are illustrated and discussed /14,15/. It is assumed that the structure functions properly in the original and altered state under static loads. The main types of failure leading to collapse of the structure during the transition stage are: (i) Deformation failure; (ii) Stress failure; (iii) Local stability failure; and (iv) Global stability failure.

No distinct difference exists between the first and second types. By the first type of failure is meant when the ductility of the material is exhausted, or e.g. a beam slipping off its supports due to large deformations. The second type relates to brittle materials, and to ductile materials made brittle by poor quality or design. By local stability failure is meant failure due to flange or web buckling. The global stability failure occurs due to column buckling and beam lateral buckling. Very distinct difference does not exist between the third and fourth cases. However, due to the difference in the order or magnitude of the masses involved, a difference in the influence of inertia forces between local and global instability exists. This difference may influence the time it takes to induce a stability failure.

In order to illustrate the various types of failures during the transition stage, consider the two-span beam in Fig. 2, where the dead load is denoted by Q, the span by L, the outer column reactions by R and the interior column force by P. The interior column is assumed to lose its load-carrying capacity according to Fig. 2.

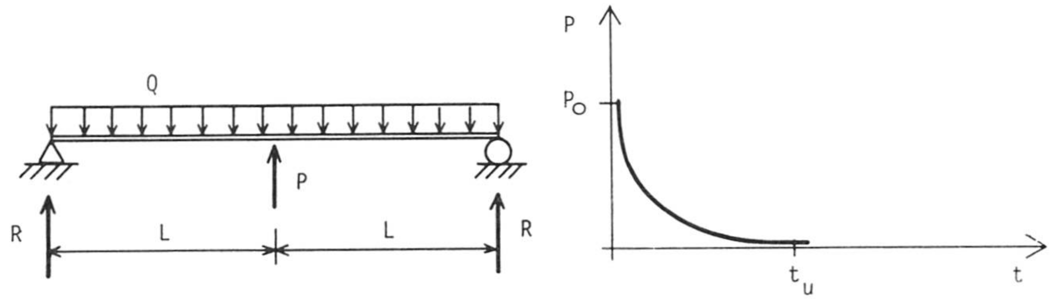

Fig. 2. Two-span beam subjected to removal of mid-support

The mid-deflection of the beam (y) versus the time (t) is shown in Fig. 3 if elasto-plastic moment-curvature relationship is assumed for the beam. Different curves are shown for different ratios of the unloading time (t_u) and the fundamental natural period of the beam (T). The maximum static deflection in the alternate path method is y_{stat} and the maximum dynamic deflection is given as y_{max} (point M). The deformation-time curve is highly dependent upon the ration t_u/T, which for practical cases is of the order of 0.1 and thus causes great dynamic effects. At point P the full plastic moment of the beam is reached (y_p) and thus the mid-moment of the beam remains constant. Even though the moment does not increase, energy for the deformation is absorbed, and the kinetic energy or velocity decreases so that the deflection reaches a maximum value, point M. Thereafter the beam turns back and revibrates under elastic conditions. However, revibration only takes place if the actual load Q is less than the maximum static load Q_p (or correspondingly that $y_{stat} < y_p$). If this is not the case, energy is releasesd during the deformation and the deflection of the beam increases continuously without limit according to Fig. 4. The asymptotic behaviour of the deflection curve is due to the assumption of pure bending moments under elasto-plastic conditions. Strain-hardening effects and catenary action constitute a considerable reserve strength, which at the same time removes the asymptotic behaviour of the deflection curve.

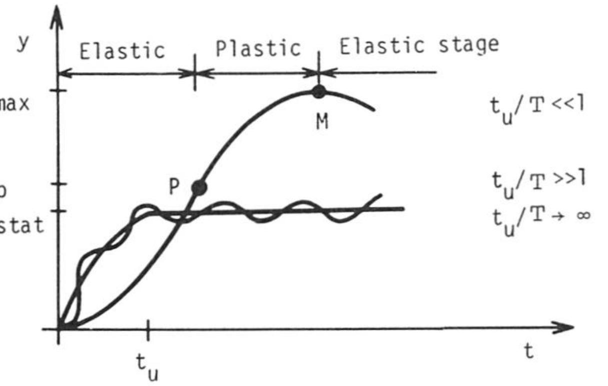

Fig. 3 Mid-deflection of a two-span beam subjected to loss of interior support

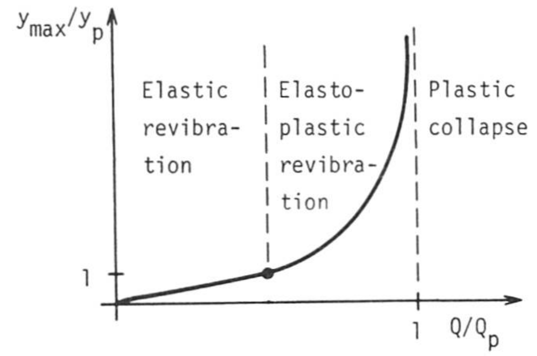

Fig. 4. Maximum dynamic deflection ratio versus the load ratio

Deformation failure may arise during the critical stage P-M in Fig. 3, when the maximum dynamic deflection y_{max} always exceeds the corresponding static deflection y_{stat}. Under elasto-plastic assumptions it is sufficient to design the beam for the alternate path under static conditions considering stress. It would not however suffice as far as the deformations are concerned. In this case the method of alternate path is applicable if the deformation capacity of the beam is adequate with regard to maximum deflection.

Steel structures are usually ductile in an ultimate stage. If, for some reason, brittle properties of the beam in Fig. 2 become significant and only small plastic deformations can be absorbed, stress failure may arise during the P - M stage, since in cirrespondence with the deflection the maximum dynamic mid-moment M_{max} will always exceed the static moment M_{stat}. Thus, the beam must be designed so that the maximum dynamic moment is less than the brittle fracture moment capacity (< M_p).

Local stability failures occur at a much higher load than the theoretical buckling load for most web and flange buckling phenomena due to the postbuckling behvaiour. The magnitude of the postbuckling range in I-beams is usually greater for the case of web buckling than for flange buckling since the web is supported on two sides while the flange only on one, Fig. 5. If the bearing capacity of the beam web in the damaged state is exploited up to point D there is a small reserve left with regard to stress but a significant reserve with regard to strain. This large deformation capacity available after "web buckling" in a critical section can be likened to the yield capacity of the material available after the yield stress has been reached. Thus, a large deformation reserve would make energy absorbtion under approximately constant load on the web possible and the conditions would be similar to those treated in the case of deformation failure. Due to the fact that the flange normally has much less deformatioin capacity, the flange buckling phenomenon is more critical than the web buckling. Since only small masses are involved in case of flange buckling the inertia forces will probably not be able to prevent stability failure during the critical P - M stage (Fig. 3) if only static design of the alternate path is made. The conditions for flange buckling are somewhat like those discussed for brittle stress failure.

By global stability failure is meant failures due to lateral buckling of the beam or buckling of the outer columns of the frame in the area of primary damage. The strain-time curve of the beam in case of no lateral buckling is directly related to the deflection-time curve in Fig. 3 and is shown in Fig. 6, denoted mid-strain. For this case the strain is uniform across the whole width of flange. Yielding occurs at ε_y. In the case of lateral buckling the strains become

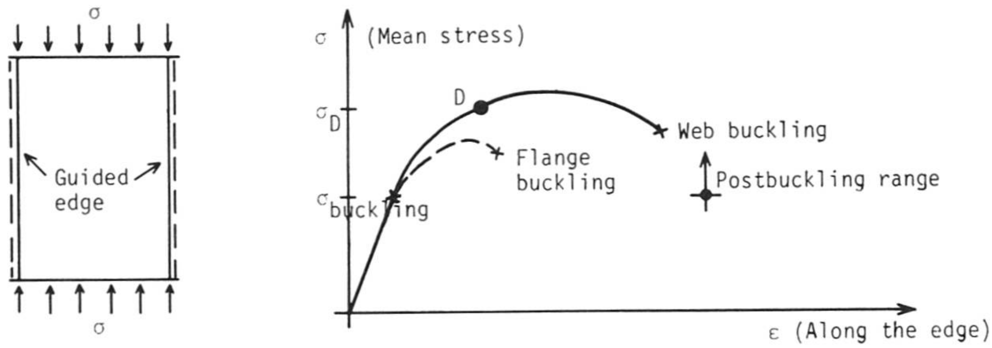

Fig. 5. Stress-strain curves for plates in compression

non-uniform in the upper beam flange and the strains at the two edges A and B are shown in Fig. 6. The curves (a) are related to large lateral deformations and the curves (b) to small ones. Simplifying the lateral buckling phenomenon to imply only pure lateral bending (without torsion) it is clear that in case (a) an unloading of the upper flange at edge B takes place and thus that the lateral stiffness of the beam in this section is determined by the reduced modulus given static conditions. However, in case (b) a loading across the whole width of the upper flange takes place and thus no lateral stiffness exists when the compressive strain exceeds the yield strain ε_y. This implies that for a certain rate dy/dt ($\Rightarrow$ $d\varepsilon_0/dt$) of the vertical deflection there exists a maximum rate du/dt ($\Rightarrow$ $d(\varepsilon_B - \varepsilon_A)/dt$) of the lateral deflection of the upper flange for which zero lateral bending resistance of the flange is possible. At a greater rate the lateral deformation would become dominant and the conditions according to case (a) would occur. However, the inertial forces may have a restraining effect upon the lateral deflection during the critical period P to M in Fig. 3. After redeflection, unloading across the whole width of the flange takes place with considerably increased lateral bending stiffness as a consequence. The method of alternate path under static conditions is unable to predict whether the beam will buckle in a lateral mode during the stage of zero lateral bending resistance. (Note: in actual cases strain-hardening effects and torsional rigidity may affect the lateral buckling resistance considerably.)

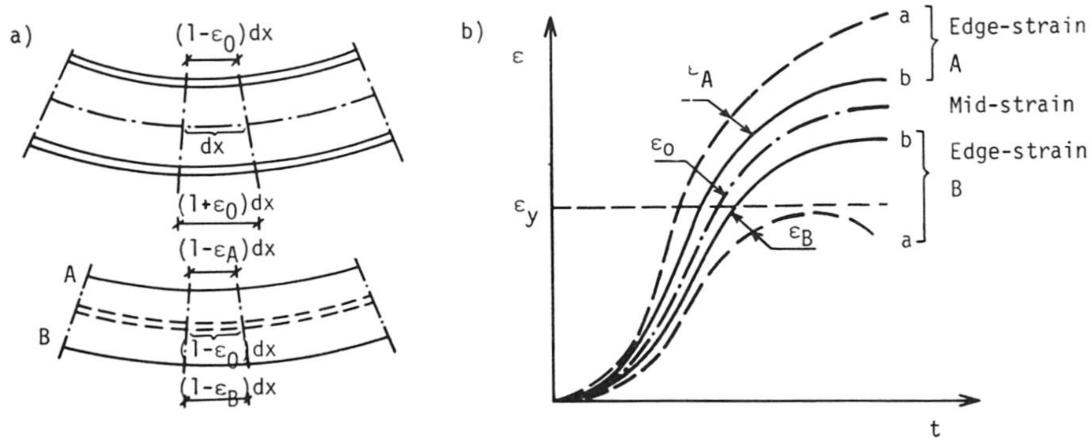

Fig. 6. Non-uniform compressive strain of the upper beam flange (a) and its time curve (b)

In the case of column buckling the maximum dynamic column force will always exceed the static column force after damage (alternate path), cf. Fig. 3. Thus, the principle of alternate path does not ensure column stability during the stage of dynamic transition. Due to initial curvature of the actual column and load excentricity, the column deflects under action of moments and thus to a certain extent behaves as the beam with regard to deformation, stress, local and global (lateral and torsional buckling) stability failures. However, the stress and strain distributions of the column are not identical to that of the beam.

An example of an actual practical case for which a collapse occurred, probably due to the dynamic effects during the stage of dynamic transition, is the steel-framed structure shown in Fig. 7 /16/. At the time when the building had just been completed and there was some, but not much, snowload on the structure, a very poor weld in the upper flange of the rigid connection at A of the beam AB failed. Consequently, the clamping moment but not the support reaction at A disappeared and the field moment of the beam AB increased. From a design point of view, the beam AB would have been able to carry the actual load under static conditions if a flexible joint at A had been assumed, but the beam collapsed probably due to the following reason: The removal of the clamping moment at A was a rapid phenomenon in principle according to Fig. 2, which caused a dynamic deformation process in principle according to Fig. 3. An estimation of the maximum dynamic deflection showed that it was significantly higher than the maximum static deflection. Under these conditions, the deformation capacity of the upper flange was insufficient, since the deformation capacity was limited by torsional buckling of the flange.

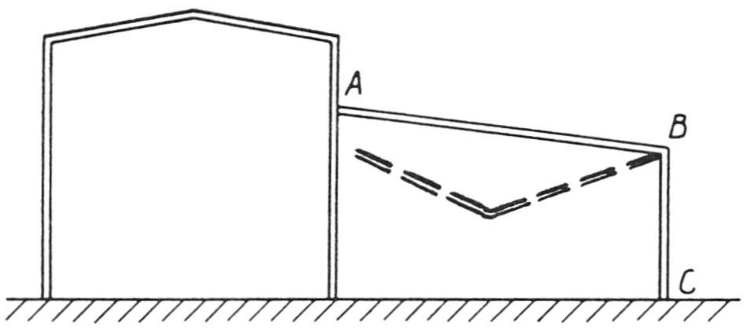

Fig. 7. Collapse of an actual steel-framed structure due to the effects of the dynamic transition stage

It is evident from the examples discussed and illustrated above that the method of alternate path under the assumption of static conditions is unable to prove the capability of a structure to survive a local damage in many cases. The stage of dynamic transition requires a different and more critical design criteria. Therefore, it is necessary that dynamic analyses and evaluations for the different cases discussed above be performed.

4. APPROACHES FOR ANALYSIS OF DAMAGE TOLERANCE

In order to analyze the capacity of a structure to survive local damage it is necessary to know the extent and time-dependence of the local damage, the properties of the materials, components, connections and structural systems etc. Many of these parameters are not known precisely. Due to the complexities and uncertainties in practical cases some simplifications are necessary. These simplifications concern the geometrical model used, the load-bearing system and strength properties assumed and the method of analysis applied.

It is not practical to investigate a variety of steel structures which represent different structural systems, beam-to-column connections and secondary load-bearing systems. Therefore, it is desirable to choose a representative structure of fundamental importance. This representative structure could be the basis of a geometrical model. It is then necessary that this model is capable of describing the essential behaviour of the actual structure. The representative structure could be analysed as a three-dimentional system or as a two-dimensional system representing the third dimension by spring parameters according to Fig. 8. Loss of column D is illustrated in Fig. 8 (only loss of an interior column will be treated here and not the special case of loss of an outer column).

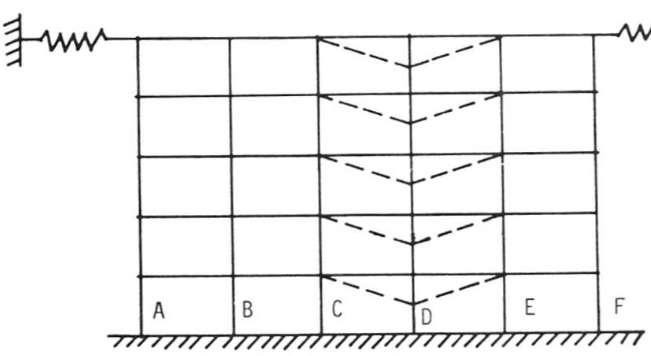

Fig. 8. Two-dimensional steel skeleton model representing the third dimension by springs

This removal can be expected to influence mainly the vertical section C-D-E. The damaging effects do not normally propagate sideways outside this section. A model representing such a section is shown in Fig. 9. The connection to the surrounding structure is represented by translational and rotational springs. These boundary forces and moments are in fact dependent on the deformations of the surrounding structure, i.e. its dynamic response. However, as a first step it is reasonable to consider only the boundary parameters owing to the force/moment-deformation characteristics of the joints at the boundary. If each floor in Fig. 9 is rigidly connected by the mid-columns to the other floors it is sufficient to consider the model in Fig. 10. This model is the fundamental one and will be used to evaluate the dynamic effects. The boundary forces and moments bear reference to the properties of the beam-to-column connections, the secondary structural system (e.g. a monolitic concrete floor) or both. As a first step, only the primary load-bearing system is considered, i.e. the steel skeleton system itself. In the second step the floor structure is considered as an attached mass without stiffness or bearing capacity. The boundary forces must, in a second step, be transferred to the remaining part of the structure and the over-all stability of the structure must be checked. A dash-pot could be added (at the mid-column) to the model in Fig. 10 to represent the energy-absorbtion capacity of interior walls. Note that the model in Fig. 10 is applicable also to continuous steel beams, e.g. in warehouse buildings and bridges.

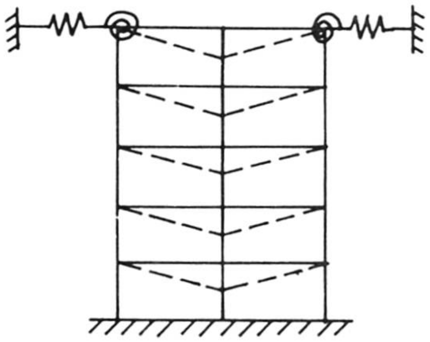

Fig. 9. Steel skeleton model in the section of local damage

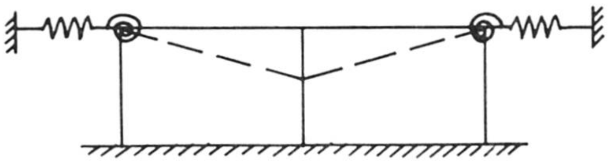

Fig. 10. Steel skeleton model in the area of local damage

The load-bearing system chosen is mainly related to bending action, catenary action or both. In the models the structural system is represented by the rotational springs (bending action), translational springs (catenary action) and the beam/column itself (bending and/or catenary action). Solely bending action often means poor exploitation of the inherent capacity of the structure but sometimes constitutes the only load-bearing system available (e.g. a simply supported two-span beam). If large deformations occur catenary action is an available and effective load-bearing system and is for certain skeleton systems the only one available (cf. Fig. 13 a).

The spring and beam/column characteristics represent the strength properties assumed. As far as steel skeleton systems are concerned there are two schools of thought regarding the design of moment connections. One approach presumes that the plastic hinges or deformations form in the parent beams, while the other approach presumes that the hinges or deformations form actually within the connections. In the first approach it follows that the spring characteristics (if the influence of the surrounding structure is neglected) as well as the beam/column characteristics are determined by the strength properties of the beam/column itself. The same principle also applies to continuous beams. The strength properties then bear upon elastic, plastic and strain-hardening conditions. In the second approach the spring characteristics are mainly determined by the properties of the connections under bending/catenary action.

Different kinds of dynamic methods of analysis exist, and these can be divided into the following groups: (i) Rigorous methods; (ii) Approximate methods; and (iii) Bounding techniques. By rigorous methods is meant application of methods for continuous mass systems while approximate methods imply application of methods where the actual continuous properties are approximated by piecewise elements or lumped masses. The bounding techniques imply determination of lower/upper bounds of certain quantities without analysing the dynamic process.

Due to the many unknown or poorly known parameters in progressive collapse problems and due to the desire to investigate the basic dynamic behaviour and capacity of steel structures subjected to removal of load-carrying elements, it is deemed adequate to study simple geometrical models (Fig. 19). These models should represent realistic characteristics of the members and joints, and permit the application of simple approximate dynamic methods of analysis.

5. MODELS OF DAMAGED STEEL STRUCTURES

These fundamental models of common real steel structures subjected to removal of an interior column in the area of primary damage are shown in Figs. 11, 12 and 13. The models are characterized by the type of load-bearing system, i.e. bending and/or catenary action, and by the kind of controlling strength properties, i.e. member and/or joint characteristics. The attached mass load is denoted Q and the destruction of the interior column is assumed to occur as a pure loss, i.e. the column support force is assumed to decrease gradually during a certain unloading time (any additional forces and moments imposed on the system during the destruction process of the column are thus neglected as in the case of the method of alternate path.

Models for continuous steel beams with different end constraints, e.g. main beams in steel bridges, are shown in Fig. 11. The model beam with ends fully restrained against axial motion is termed as a hinged beam. End constraints arising in such beams are due to beam continuity and/or end support conditions. In Fig. 12, models of steel frames with different kinds of beam-to-column connection characteristics are shown. The common bolted end-plate connections are considered in particular. The model beam with ends fully restrained against moment rotation and axial motion is termed as a clamped beam. Models for steel skeleton systems with simple supported beams are shown in Fig. 13. The very common system used in Sweden with bolted heel connections is considered in particular. It is evident that no model systems exist for load-bearing systems with pure bending action.

These models have been analyzed and the dynamic effects evaluated for different load-bearing systems and strength properties in the study /17/ and will be presented to the international research community in subsequent papers.

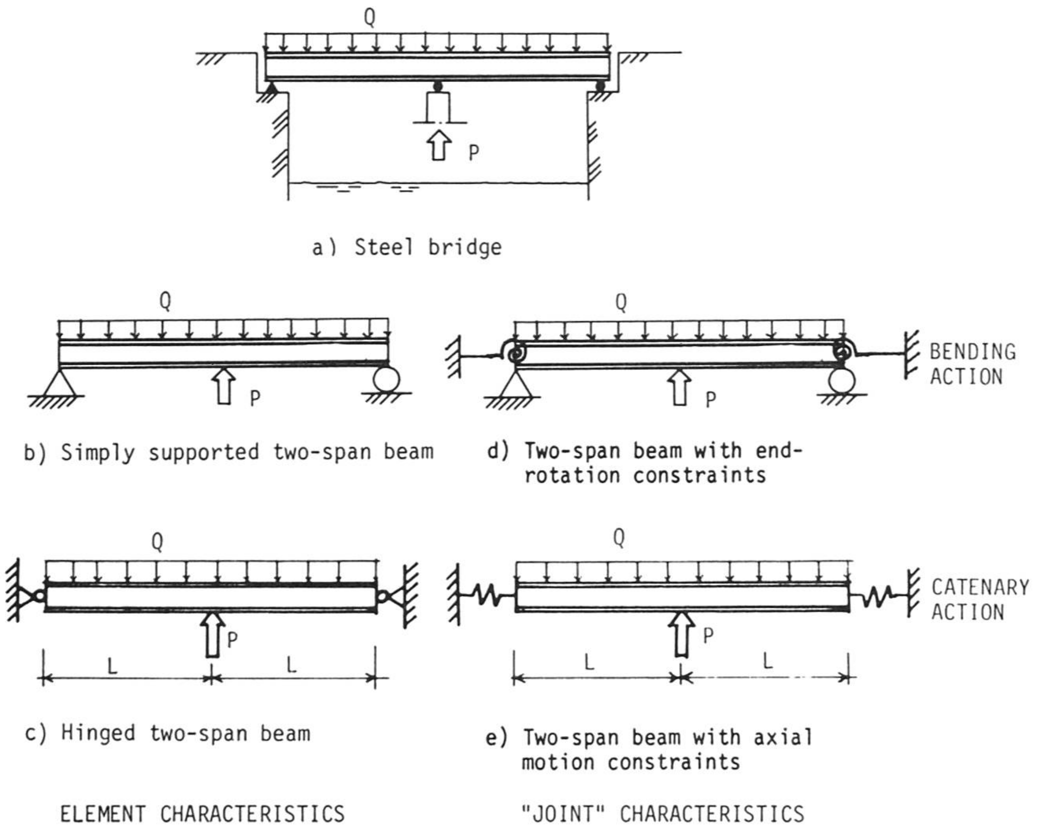

Fig. 11. Steel bridge and its models

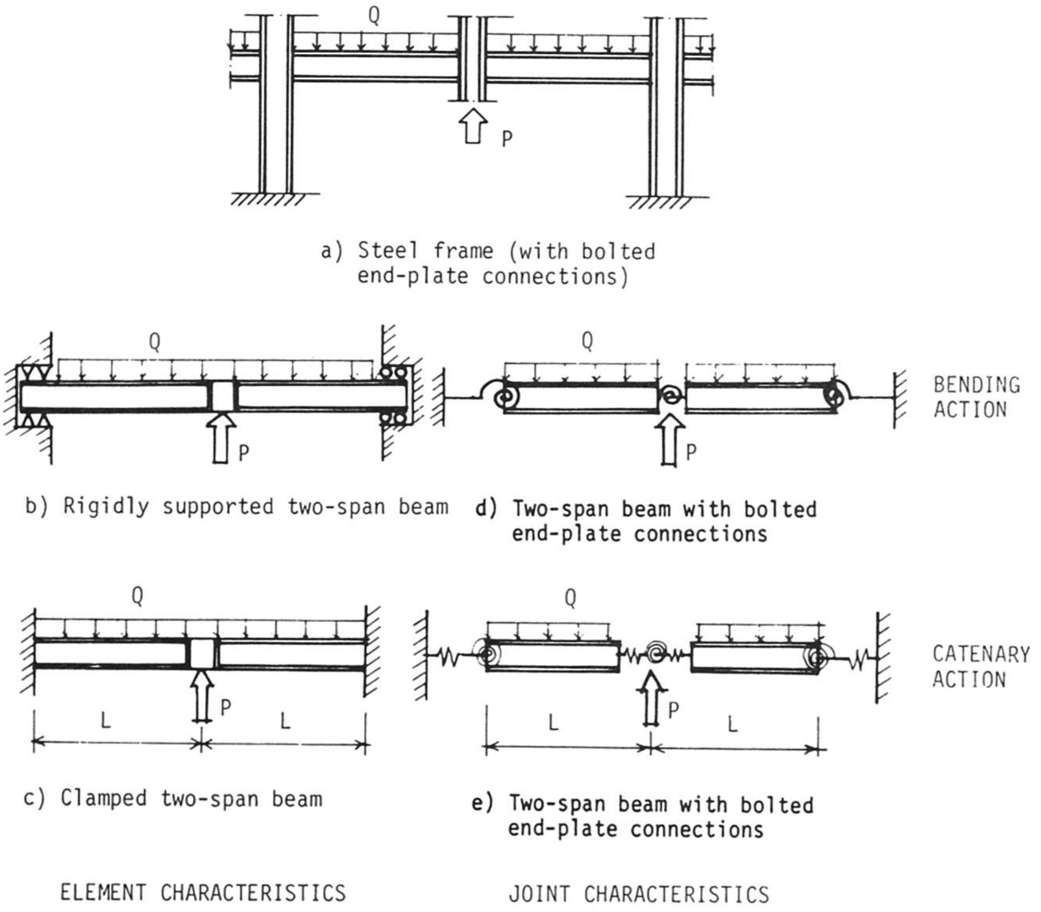

Fig. 12. Steel frame and its models

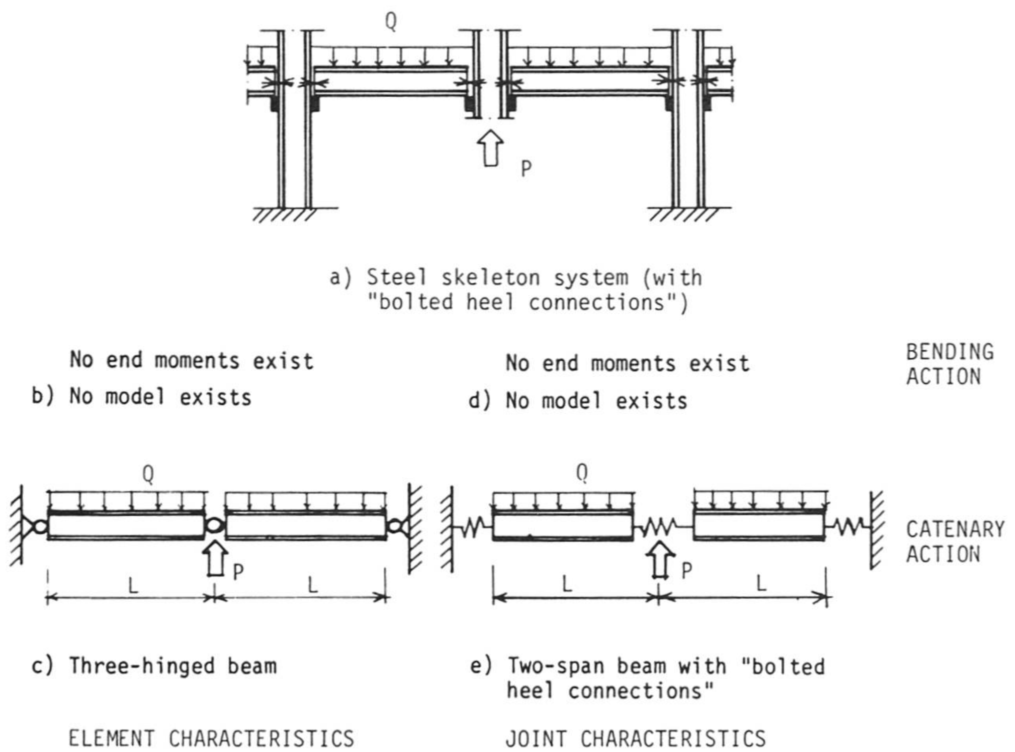

Fig. 13. Steel skeleton system and its models

SYNOPSIS

Large panel structures are in general more susceptible to progressive collapse than steel, timber and other framed structures. However, the problem of progressive failure is of a general nature and should be taken into account in all types of buildings regardless of material, type of structure, and construction method used. Design principles which preclude progressive collapse proposed in most countries are essentially based on static considerations and the method of alternate load path. However, since many types of abnormal loads are rapid phenomena, dynamic effects cannot be disregarded. The capacity of the structure during the stage of dynamic transition between the original and the damaged systems must be taken into account. In this paper, the basic design principles, the fundamental behaviour and capacity of steel structures subjected to rapid removal of interior load-bearing columns have been discussed.

The main types of failures leading to collapse of the structure during the stage of dynamic transition are deformation, stress and local/global stability failures. The basic dynamic effects on these failure types are illustrated in the paper and it is concluded that the method of alternate path under the assumption of static conditions is in many cases unable to prove that the structure is capable of surviving a local damage. The phenomenon of dynamic transition probably requires different and more critical design criteria.

Approaches for analysis of damage tolerance of structures and models of damaged steel-framed structures have also been discussed. The complexity of actual structures subjected to abnormal loads necessitate some simplifications for analysis purposes. These simplifications relate to the geometrical model used, the load-bearing system and strength properties assumed and the method of analysis applied. The most common types of steel structures and their corresponding geometrical models have been presented. Only interior support removal and the behaviour in the area of primary damage are considered. Both bending action and catenary action of the models are considered. Also, the strength properties of the members and of the beam-to-column connections are considered in the models.

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