Cumul de déformations plastiques par charges répétées

Inkrementale plastische Deformationen infolge wiederholter Lasten

Incremental Plastic Deformations by Repeated Loads

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Dans les exposés qui ont été présentés, on a vu l'action de charges répétées alternatives provoquant dans certaines sections des plastifications alternées, qui donnent lieu à des cycles dont chacun correspond à une énergie dissipée.

Le caractère principal est que dans chaque section les déformations de plastification sont alternativement dans un sens ou dans le sens opposé : allongement plastique suivi d'un accourcissement plastique, courbure plastique dans un sens suivie d'une courbure plastique dans le sens opposé.

Nous nous référons ici à des *actions toujours de même sens*. Mais ce sont des actions répétées successivement, soit que l'on ait des actions indépendamment variables, soit que ces actions correspondent à des charges mobiles.

On peut voir apparaître alors des déformations plastiques cumulatives. Mais, contrairement au cas précédent : *les déformations plastiques ont lieu dans des sections différentes et dans chaque section les déformations plastiques sont toujours de même sens et cumulatives.*

Nous allons donner deux exemples : l'un se référant à deux forces indépendamment variables, l'autre à une charge mobile.

I - ACTIONS DE DEUX FORCES INDEPENDAMMENT VARIABLES

Un portique ABB'A', biarticulé en A et A', est soumis alternativement à une action verticale $\lambda_1 P$ au milieu C de BB' et à une action horizontale $\lambda_2 H$ en B.

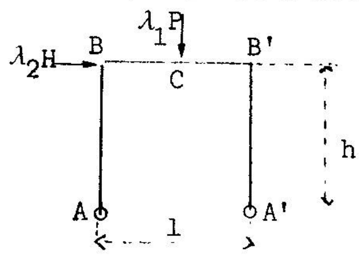


Fig. 1

On prend $l = h = 1$. Les inerties de toutes les barres ABA'B' sont égales à l'unité. Il y a une inconnue hyperstatique : la poussée Q.

I.1 - Action de $\lambda_1 P$ seule

Dans le domaine élastique :

$$Q = \frac{3}{40}\lambda_1 \qquad M_B = M_{B'} = -\frac{3}{40}\lambda_1 \qquad M_C = +\frac{7}{40}\lambda_1$$

Toutes les barres ont même section. Soit M_o le moment de rotule plastique.

Les conditions de plasticité s'écrivent :

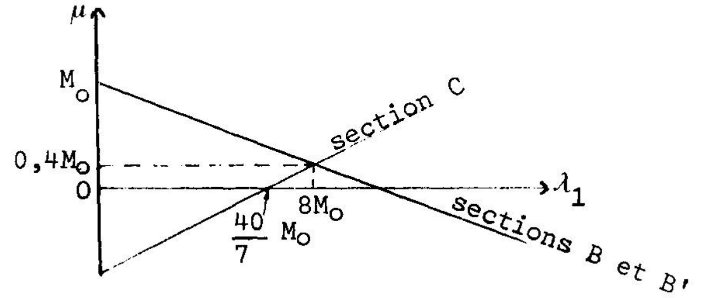
$$-M_o \leqslant -\frac{3}{40}\lambda_1 - \mu \leqslant +M_o$$

$$-M_o \leqslant \frac{7}{40}\lambda_1 - \mu \leqslant +M_o$$

μ étant le scalaire qui, appliqué à la force unité, donne *la poussée Q d'autocontrainte.*

En se bornant au domaine $\lambda_1 \geqslant 0$, on a le diagramme de Rjanitsyn.

Fig. 2



Charge limite élastique : $\lambda_{1e} = \frac{40}{7} M_o$.

Par dépassement, il y a plastification en C.

Charge ultime : $\lambda_{1u} = 8\ M_o$.

(M_o est un scalaire moyennant les conventions d'unité).

L'application de $\overrightarrow{\lambda_{1u} P}$ donne plastification en C - autocontrainte Q = 0,4 M_o.

I.2 - Action de $\lambda_2 H$ seule

Les conditions de plasticité s'écrivent :

$$-M_o \leqslant \frac{\lambda_2}{2} - \mu \leqslant +M_o$$

$$-M_o \leqslant -\frac{\lambda_2}{2} - \mu \leqslant +M_o$$

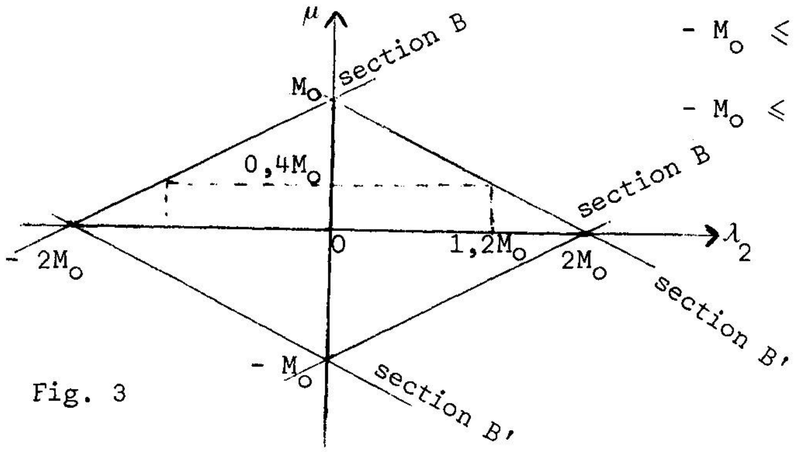
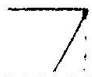


Fig. 3

La charge ultime λ_{2u} = 2 M_o correspond à une autocontrainte nulle.

Une autocontrainte 0,4 M_o provoque une plastification en B' lorsque λ_2 dépasse la valeur de 1,2 M_o.

I.3 - Actions alternées de $\lambda_{1u}P$ et $\lambda_{2u}H$

	$\lambda_1 P$	$\lambda_2 H$	Plastification en C	Plastification en B'	Autocontrainte Q
1	$\lambda_1 = \lambda_{1u}$	$\lambda_2 = 0$			0,4 M_o
2	$\lambda_1 = 0$	$\lambda_2 = \lambda_{2u}$			0
3	$\lambda_1 = \lambda_{1u}$	$\lambda_2 = 0$			0,4 M_o
4	$\lambda_1 = 0$	$\lambda_2 = \lambda_{2u}$			0

Plastifications toujours de même sens, tantôt en C, tantôt en B'.

Après chaque action $\lambda_{1u}P$: autocontrainte 0,4 M_o.

Après chaque action $\lambda_{1u}H$: autocontrainte nulle.

II - CHARGE MOBILE

Nous renvoyons à l'étude que nous avons présentée au Congrès de New-York en 1968 : "Optimisation des structures par la considération des états limites plastiques".

Une charge mobile sur une poutre continue à deux travées égales et de section constante provoque des plastifications, tantôt en travée (courbures en concavité vers le haut), tantôt sur l'appui central (courbure à concavité vers le bas).

Il y a plastifications cumulées toujours de même sens dans des sections différentes, avec variation à chaque fois de l'autocontrainte.

III - CONTROLE DU NON-CUMUL DE DEFORMATIONS PLASTIQUES SOUS CHARGES ULTIMES

La structure est soumise à p systèmes d'actions indépendamment variables

$$\lambda_1 \vec{A_1} \qquad \lambda_2 \vec{A_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad \lambda_p \vec{A_p}$$

(on peut également traiter ainsi les charges mobiles, avec un nombre suffisant p de positions possibles).

On aura des charges ultimes : λ_{1u} λ_{2u} .... λ_{pu}, qui ne provoquent pas de cumul s'il est possible de trouver au moins un système $\vec{Q}$ d'autocontraintes, tel que : $F(\lambda_i \vec{A_i}) + \vec{Q}$ respectent en tous points les conditions de plasticité, $F(\lambda_i \vec{A_i})$ étant toutes les sommes $\sum_i \lambda_i \vec{A_i}$, où λ_i prend pour chaque valeur de i de 1 à p les valeurs 0 ou λ_{iu}.

RESUME

On traite ici le cas de charges toujours de même sens mais répétées, soit qu'elles soient indépendamment variables, soit qu'elles soient mobiles. Les plastifications toujours de même sens se produisent successivement dans des sections différentes, engendrant par répétition cumul de déformations plastiques. On vérifie que l'on n'a pas cumul de déformations plastiques s'il existe au moins un système d'autocontrainte compatible avec toutes les charges ultimes.

ZUSAMMENFASSUNG

Der Aufsatz behandelt den Fall gleichgerichteter, wiederholter Lasten, die entweder unabhängig veränderlich, oder beweglich sein können. Die stets gleichgerichteten Plastifizierungen stellen sich nacheinander in verschiedenen Abschnitten ein, woraus durch Wiederholung inkrementale Deformationen entstehen. Es wird nachgewiesen, dass sich keine plastischen Deformationen bilden, wenn ein System von "Selbstbeanspruchung" existiert, das mit allen Lastfällen verträglich ist.

SUMMARY

This paper deals with the case of repeated loadings acting always in the same direction. The applied loads are either independently variable or movable. Plastification occurs always in the same sense in different sections, which creates incremental deformations. We verify that we don't have incremental deformations if we have a system of "auto-stress" compatible with all loading cases.

Summary Report on Theme I

Rapport sommaire au thème I

Zusammenfassender Bericht zum Thema I

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1. Comments on the Prepared Discussions

1.1 Material Behaviour Under Cyclic Stressing

Theory for load-deformation behaviour must of necessity be based on the stress-strain characteristics of the materials. At the present time theory for cyclic load behaviour is based on simple models of stress-strain behaviour because of lack of more detailed experimental data and because in some cases the use of complicated models is not warranted.

Three contributions to the prepared discussion were concerned with the stress-strain behaviour of metals. Lind and Mróz use Masing-type stress-strain relationships for metals with complex stress states in the plastic range to obtain a simplified theory for small deformation behaviour of structures under cyclic proportional loading. Ellyin and Neale present test results for an aluminium subjected to cyclic biaxial stresses in the plastic range which give information on cyclic strain hardening and shear-direct stress interaction curves. Kato and Akiyama present a method for obtaining the stress-strain curve for uniaxial cyclic (reversed) loading of steel from the monotonic loading curve by a procedure which involves displacing part of the monotonic curve horizontally. These three contributions are a welcome addition to existing knowledge.

The method suggested by Kato and Akiyama for cyclic stress-strain curve determination of steel gives a convenient approach but is largely intuitive and as such can only be accepted after thorough experimental confirmation which is as yet lacking. For steel subjected to uniaxial cyclic loading it does seem that stress-strain curves based on Ramberg-Osgood type expressions, with the coefficients found experimentally, gives an accurate stress-strain model.

It is evident that much more comprehensive experimental data is still required for metals, particularly steel, under cyclic (reversed) loading with different strain rates, and under simple and complex stress conditions.

There were no contributions on the stress-strain behaviour of concrete to the prepared discussion of this Theme. However, previous tests on concrete subjected to uniaxial repeated compressive loading have indicated that the envelope stress-strain curve is practically identical to the stress-strain curve obtained from a single load application. Simplified stress-strain curves for repeated loading can be postulated within this envelope curve which in many cases are sufficiently accurate for accurate moment-curvature calculations. However, once again more comprehensive experimental data is required, particularly on the complete stress-strain curve for repeated compressive loading of concrete confined by transverse steel reinforcement, for rapid strain rates, and for complex stress situations.

1.2 Theory for Structural Steel Elements and Structures

There were six contributions to the prepared discussion concerned with the theoretical behaviour of structural steel elements and structures. Chen studies the moment-curvature relations for steel H columns under repeated and reversed compression combined with biaxial bending moments. He shows that the Ramberg-Osgood moment-curvature relationship is highly satisfactory, fitting well with theory, and observes a transformation of the yield surface. Burth and Vogel determine theoretically the load-deflection relationships of two rectangular portal frames up to the ultimate load using second order elasto-plastic theory for a single load application. The effect of order of loading on the load-deflection curves is illustrated. Dimitrov and Glaser determine theoretically the ultimate load of a vibrating column. Wada, Suto and Fujimoto give a non-linear analysis of K-type braced steel frames subjected to repeated horizontal loading. This analysis takes into account the non-linearities due to yield of the steel and change of geometry of the structure due to deflections, and good agreement with experimental results is found. Kato and Akiyama extend their displaced curve method (referred to previously for stress-strain determination) to members and frames. The method calculates cyclic load-deflection relationships from the monotonic curves. Apart from a lack of roundness of the curves (equivalent to ignoring the Bauschinger effect) good agreement between the constructed and experimental curves is obtained. Yokoo, Nakamura, Ishida and Nakamura present a method for computing static and dynamic load-deflection curves for frames with alternating horizontal loading, taking into account non-linear stress-strain characteristics and the change of geometry due to deflections. The method appears to be extremely powerful and valuable for realistic theoretical studies of the effect of seismic motions on frames.

The above contributions cover a wide area and represent a valuable step forward. They indicate that sophisticated theoretical analyses can be developed to follow through in detail structural steel behaviour under repeated and reversed loading. It is evident that for such analyses realistic moment-curvature characteristics of members are required and the effect of change of geometry due to deflections needs to be included (the P-Δ effect). Also the stiffness of connections and panel zone deformations may need to be included. It is to be noted that previous work by Popov and Bertero, reported at the 5th World Earthquake Conference, Rome 1973, has indicated how panel zone deformation may be included in a theoretical analysis, using a moment-shear deformation model for the panel zone.

1.3 Theory for Reinforced Concrete Elements and Structures

There were five contributions to the prepared discussion in this area.

Singh, Gerstle and Tulin observe that the shear strength of beams may decrease with cyclic loading because bond deterioration leads to aggregate interlock cracking, to dowel cracking and to more extensive diagonal tension cracking. Thus cyclic loading can reduce the shear strength of members as well as causing additional deformations in the plastic hinge regions. This points to the necessity of providing adequate shear reinforcement to carry the majority of the shear force by truss action in members subjected to intense cyclic loading.

Menegotto and Pinto calculate theoretical load-deflection curves for cyclically loaded reinforced concrete frames using the stress-strain curves for the steel and concrete to obtain the load-moment-rotation characteristics of the members, and a step-by-step numerical procedure to follow through the deflection behaviour. Changes of geometry due to deflections are taken into account but the effect of bond slip and shear deformation is ignored. Good agreement is obtained with the experimental results obtained from a frame. Muguruma, Tominaga and Watanabe present experimental results and a model for taking account of the additional deformations due to shear and bond slip in columns. McClure, Gerstle and Tulin show analytically that creep of concrete should be included, as well as strain hardening, when assessing the incremental deformations of concrete structures subjected to cyclic overloads. Good correlation is obtained between their theory and tests. Gluck presents an analytical procedure for determining the ductility of coupled shear walls. This procedure is particularly useful for assessing the required rotational ductility factor of the coupling beams of shear walls for a given overall ductility factor for a single load application. These papers compliment each other and indicate that theoretical analyses and experimental data on which to base models are becoming available to allow determination of the load-deformation behaviour of complete reinforced concrete frames and walls under monotonic and cyclic loading.

2. Concluding Remarks

2.1 Theoretical Approaches

It is evident that the theoretical procedures are available which are capable of following in detail the behaviour of frames responding to cyclic loading, and that for this theory to be accurate it must be based on realistically shaped moment-curvature loops, taking into account possible stiffness and strength degradation.

For structural steel, Ramberg-Osgood type functions give a good indication of the stress-strain and moment-curvature curves for cyclic loading. For reinforced concrete, the shape of the moment-curvature curves may be predicted accurately from the stress-strain curves for the steel and concrete providing that the shear force is not high and that significant bond deterioration does not take place. For reinforced concrete beams with equal top and bottom steel the moment-curvature loop for cyclic loading is strongly influenced by the shape of the stress-strain curve for the steel and therefore closely resembles a Ramberg-Osgood type loop. For such beams full depth open cracks will exist down the section for most of the loading range. For beams with different steel areas top and bottom, and for columns, the moment-curvature loops for cyclic loading show a marked pinched in shape due to changes in stiffness caused by closing of cracks in the compression zone when the compression steel yields. The shape of such loops can be derived from first principles or assumed to follow the shape of a model, for example the degrading stiffness response suggested by Clough. It is to be noted that for reinforced concrete beams generally the ultimate moment capacity is not significantly effected by the reduced stiffness but is reached at greater deflections. For columns, however, loss of section due to crushing of concrete will reduce the moment capacity and cause some strength degradation. For prestressed concrete, accurate theoretical moment-curvature curves for cyclic loading can be derived which indicate that prestressed concrete members show a high recovery of deformations and, because of narrow hysteresis loops, little energy may be dissipated. At large deformations when the member becomes damaged the energy dissipation becomes much greater.

In frames, deformations in addition to those caused by flexure may be important. These additional deformations may be due to alternating shear forces in members leading to shear deformations, particularly at plastic hinge regions, alternating shear forces in panel zones of joints causing shear deformations, and alternating bond forces in concrete members leading to a deterioration of bond and to deformations due to slip of steel. The effect of change of geometry due to deflections may also be important in many cases, especially for structural steel frames.

2.2 Ultimate Deformability

The definition of ultimate deformability of an element or a structure often causes difficulty. The ultimate deformability is not necessarily that deformation corresponding to the maximum load carrying capacity, and hence it is not necessarily that deformation which causes an extreme fibre compressive concrete strain of 0.0035, or thereabouts, or a tension steel strain of about 0.01. It does seem that the definition adopted for ultimate deformability should depend on how much reduction in load carrying capacity can be tolerated and/or the degree of damage to the structure which can be tolerated. If damage cannot be tolerated at all, such as in prestigous structures or structures containing dangerous chemicals, it may be that an entirely elastic response needs to be assured and the design strengths need to be correspondingly high. At the other end of the scale, survival without collapse may be the criteria in which case very large strains could be reached, and the structure damaged, during the response. For cases in between, limiting strains or deformations could be set. Many elements and structures have a capacity for deformation beyond the peak of the load-deflection curve. In cases where survival without collapse is the criteria, it is too conservative to define the ultimate deformability as the deformation corresponding to the maximum load carrying capacity. It would seem reasonable to recognize at least some of this deformation after the maximum load has been reached and to define the ultimate deformability as that deformation when the load carrying capacity has reduced by some arbitrary amount after maximum load. For example, a 10% or 20% reduction in maximum load carrying capacity could be tolerated in many cases, but the exact amount would depend on the particular case. In the case of seismic design a step-by-step dynamic analysis would be required to determine whether a certain load-displacement hysteretic behaviour of the structure was adequate to survive the earthquake.

2.3 Future Research

The previous sections have indicated areas where future research is required. These involve more adequate stress-strain models for material behaviour, and further improvements in existing theoretical procedures for the cyclic load-deflection response of complete structures. These improvements to theoretical procedures require account to be taken of deformations due to flexure, shear and bond in the general case. Thus more general models of shear and bond deformations in members and panel zones need to be determined, and these require more experimental evidence. Some such experimental evidence has been presented in Theme III.

The three dimensional behaviour of structures also requires investigation. Earthquake and wind loading seldom act in the direction of one principal axis of a building. Loading in a general direction to the principal axes of a building will cause biaxial bending of columns and bending of both sets of beams simultaneously. Thus the flexural strength of the columns will be decreased due to biaxial bending, and the total resultant flexural strength of the beam system will be increased (for example, by 41% in the case of beams of equal strength on a square grid with loading along a diagonal of the floor). In a framed building structure this could lead to brittle plastic hinges in the columns rather than ductile plastic hinges in the beams. The effect of general loading direction has so far been neglected by designers in seismic design and this aspect requires theoretical attention in the future. It does appear that diagonal earthquake loading will produce a more critical condition in the columns of a building structure than loading along one principal axis of the building.

The availability of accurate theory for cyclic load-deflection characteristics of structures and elements will no doubt lead to more realistic analyses in the future of the effect of severe vibratory motions, such as from earthquakes, on a range of structural systems. It should also be borne in mind that repeated inelastic deformations may cause severe damage to structures. The level of tolerable damage, and the extent of repair necessary to structural and non-structural elements, are factors which also need investigation.

SUMMARY

The contributions to the prepared discussion of Theme I: Theorization of Structural Behaviour with a View to Defining Resistance and Ultimate Deformability are commented upon briefly. Concluding remarks on theoretical approaches are made and areas requiring further research are indicated.

RESUME

Les contributions à la discussion préparée au thème I sont brièvement commentées. On fait des remarques de conclusion aux approchements théoriques et on indique les domaines où des recherches sont désirables.

ZUSAMMENFASSUNG

Die Beiträge zur vorbereiteten Diskussion zum Thema I werden kurz kommentiert. Man macht schlussziehende Bemerkungen über theoretische Näherungen und bezeichnet Gebiete, auf denen weitere Forschungen angezeigt sind.

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