Special Lecture

On the Non-linear Response of Ship Structural Elements

by

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1. Introduction

In this report, structural strength of ship structural elements such as plates,columns, girders and structural components which compose ship structures, are briefly introduced especially with regard to the nonlinear structural response of these elements and components, including recent topics in this field.

And an emphasis is paid to some items which are not always described in the ordinary text books.

As you well know, the research fields of ship structural strength are spread so widely that this report covers only a portion of that. In order to give you an idea about the research field, the 12 Technical Committees organized in the ISSC (International Ship Structures Congress) are listed below-

Comm. Ⅰ. 1 Environmental Conditions
Comm. Ⅰ. 2 Derived Loads
Comm. Ⅰ. 3 Design Loads
Comm, Ⅱ. 1 Linear Structural Response
Comm. Ⅱ. 2 Nonlinear Structural Response
Comm. Ⅱ. 3 Transient Dynamic Loadings and Response
Comm. Ⅱ. 4 Steady-State Dynamic Loadings and Response
Comm. Ⅲ. 1 Ferrous Materials
Comm. Ⅲ. 2 Nonferrous and Composite Materials
Comm. Ⅲ. 3 Fabrication and Service Factors
Comm. Ⅳ. 1 Computation Means
Comm. Ⅴ. 1 Design Philosophy, Criteria and Procedure

2. Design Criteria for Structural Elements

2-1 Design Methods

Structural design
- (a) Allowable stress design (Elastic design) {Optimum design
- (b) Plastic design
  - Simple plastic design (Hinge collapse design)
  - Minimum weight design
  - Shake down design

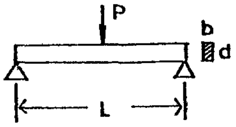
Limit (state) design
Ultimate Strength design

(i) Buckling stength design
(ii) Fatigue strength design
(iii) Brittle fracture design(Low temperature)
(iv) Creep design (High temperature)

2-2 Allowable stress Design and Plastic Design

Allowable stress design has been widely used and most traditional method and is based on the criterion that stresses induced in a structure subjected to external loads, should always be kept less than a given allowable stress level.

For example, in case of a simply supported beam with rectangular section subjected to a concentrated load at the midspan, as follows;



$$M_{max} = \frac{PL}{4}$$

$$\therefore \sigma_{max} = \frac{M}{S} = \frac{M}{I/y} = \frac{3PL}{2bd^2}$$

$$\sigma_{max} \leq \sigma_{all} \text{ (given)}$$

Fig. 1.

That is, σ_{max} should be less than the allowable stress σ_{all}.

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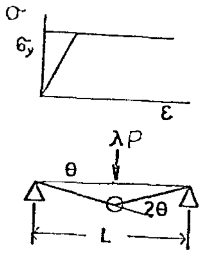
Therefore, the scantling of the beam can be determined by the condition.

On the contrary, in case of plastic design the scantling of the beam is determined in such a way that the beam is just collapsed when the load reaches λP, where λ=load factor.

In other words the ultimate strength of a structure is first calculated taking into account of the plastic behaviour of the structure, and this ultimate load is taken as the design criterion in stead of σ_{all}. In case of the above example, meterial is assumed to be perfectly elastic-plastic, then the collape load is obtained by using Upper Bound Theorem,



$$\lambda P\frac{L\theta}{2}=2M_p\theta$$

$$\therefore M_p=\frac{\lambda PL}{4}=\frac{bd^2}{4}$$

Fig. 2.

From this equation, the scantling of the beam can be determined

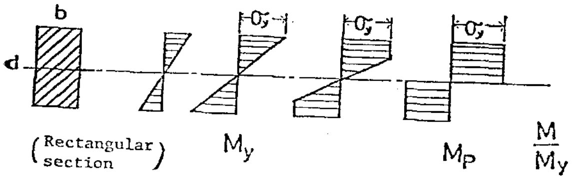


Fig. 3. Stress distribution as load increases.

This M_p is the maximum moment of which this beam can carry and is called as "Full Plastic Moment" of the beam. When the moment of a portion of a member reaches M_p, the portion is called as "Plastic Hinge", because this portion can not carry any more moment beyond M_p. As external loads increase proportionally, plastic hinges are formed in a structure successively, and then finally the structure becomes unstable, i.e., collapse. The method in which this collapse load is calculated by making use of plastic hinges, is so called "Simple Plastic Design",

Three conditions at collapse
Equilibrium Condition — Lower Bound Theorem
Plasticity Condition — Lower Bound Theorem
Mechanism Condition — Upper Bound Theorem

Let's take the same example(Fig. 4).

Now, we assume $\sigma_B=42\,\mathrm{kg/mm^2}$, $\sigma_y=28\,\mathrm{kg/mm^2}$ and $\sigma_{all}=14\,\mathrm{kg/mm^2}$, then, the factor of safety(F.S.)

$$=\frac{\sigma_B}{\sigma_{all}}=\frac{\sigma_B}{\sigma_y}\cdot\frac{\sigma_y}{\sigma_{all}}=\frac{1}{r}\cdot\frac{\sigma_y}{\sigma_{all}}$$

where, the yield ratio $r=\sigma_y/\sigma_B$ 0.6 0.65(for Mild Steel), so that

$$\text{(F.S.)}\quad \text{elastic}=\frac{\sigma_B}{\sigma_{all}}=\frac{42}{14}=3.0 \text{ or } \frac{\sigma_y}{\sigma_{all}}=2.0$$

This means that the factor of safety of three beams (Cylindrical, Rectangular and I-shape) is 3.0 for σ_B, or 2.0 for σ_y. However, you may notice that the ultimate moment of cylindrical beam is 1.7 and 1.5 for rectangular beam, while approximately 1.14 for I-shape. (Fig. 4) Can we say that these three beams have the same factor of safety? This is a sort of paradox in so-called elastic design and you may see another typical example by comparing the collapse loads of two cases, i.e., all edges simply supported and all edges fixed in Table 1.

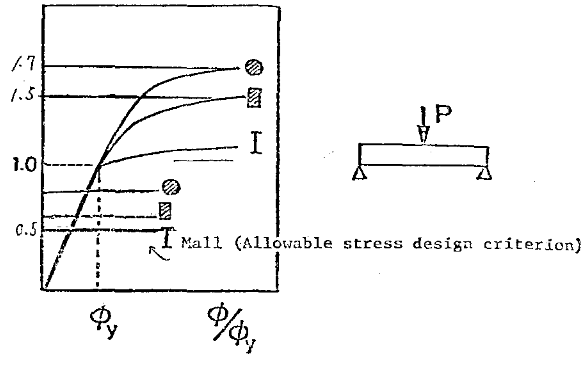


Fig. 4.

On the contrary, when we use the plastic design or ultimate strength design as the design basis, these could be no such a paradox and we can easily understand the meaning of the factor of safety, in other words, when λ(loadfactor)=2.0 and design load=10 Ton, the collapse load should be 10×2=20 Ton.

Every welded structure should have residual stresses

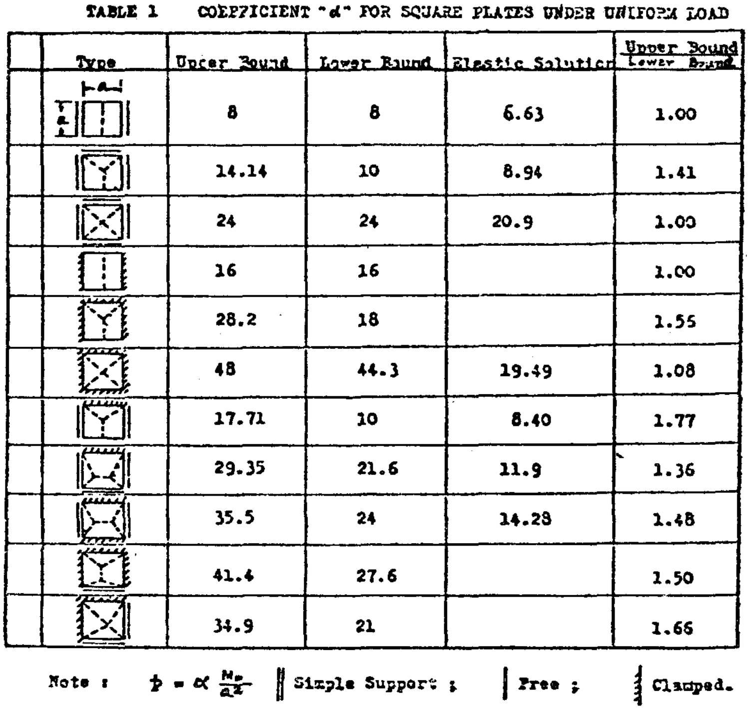


TABLE 1 COEFFICIENT "α" FOR SQUARE PLATES UNDER UNIFORM LOAD

	Type	Upper Bound	Lower Bound	Elastic Solution	Upper Bound / Lower Bound
		8	8	6.63	1.00
		14.14	10	8.94	1.41
		24	24	20.9	1.00
		16	16		1.00
		28.2	18		1.55
		48	44.3	19.49	1.08
		17.71	10	8.40	1.77
		29.35	21.6	11.9	1.36
		35.5	24	14.28	1.48
		41.4	27.6		1.50
		34.9	21		1.66

Note : $p = \alpha \frac{M_p}{a^2}$ ‖ Simple Support ; | Free ; ┆ Clamped.

Fig. 6.

due to welding which often reach to the yield stress level σ_y. Allowable stress design may not always take into account the influence of residual stresses, but in general the residual stresses have no influence on the ultimate strength of the structure, i.e., plastic design, except compressive strength of the structure. However, it should be kept in mind that there are some disadvantage in plastic design method as follows.

(1) This method does not calculate stress distribution in the structure, so that stress analysis may be required to some extent.

(2) It is rather difficult and troblesome at present to calculate the ultimate strength of large plate structure. And keen efforts have been devoted to study this kind of application program for complexed structure.

Conclusively, (1) today, structures should firstly be designed by stress analysis including checking on stress concentration and fatigue, then the important part of structure should be checked by plastic analysis in order to make sure the enough reserve strength for the unexpected over loading.

(2) Tomorrow, structure should firstly be designed by plastic analysis thoroughly, resulting in the most economical distribition of scantling, then the suspici-

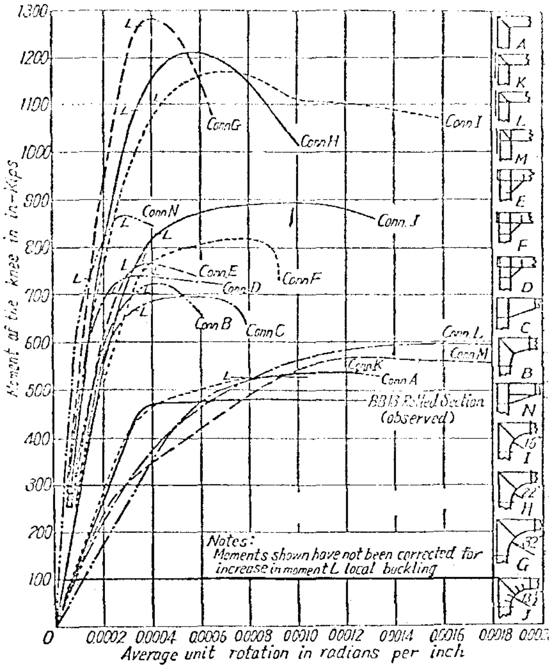


Fig. 7. connection beharior compared with 8B13 rolled section(based onmoment at the knee)

ous parts of the structure such as the portion where higher stresses are expected, for example, the part of structural discontinuities and around the holes in plate element etc., should be checked by stress analysis in order to present excess stress raise which may cause cracks.

2-3 Rotation Capacity

In the simple plastic design, it is assumed that the plastic hinges formed in a structure should keep their full plastic moment M_p up to the collapse load of the structure. In order to maintain this assumption, the local buckling of flange and web should be successfully prevented during and after the forming of plastic hinges. Fig. 7 shows that there are three categories of corner connections and direct connections such as A, K, L and M are preferable for steel skeletons subjected to static load only.

2-4 Minimum Weight Design

Let's take an example shown in Fig. 8

$$G_i=CM_{pi}^{\alpha} \tag{1}$$

$$W=C(L_1M_p^{\alpha}+\cdots\cdots+L_nM_{pn}^{\alpha}) \tag{2}$$

$$G_i=a-bM_{pi} \tag{3}$$

$$W_i=\sum G_iL_i=a\sum L_i+b\sum M_{pi}L_i \tag{4}$$

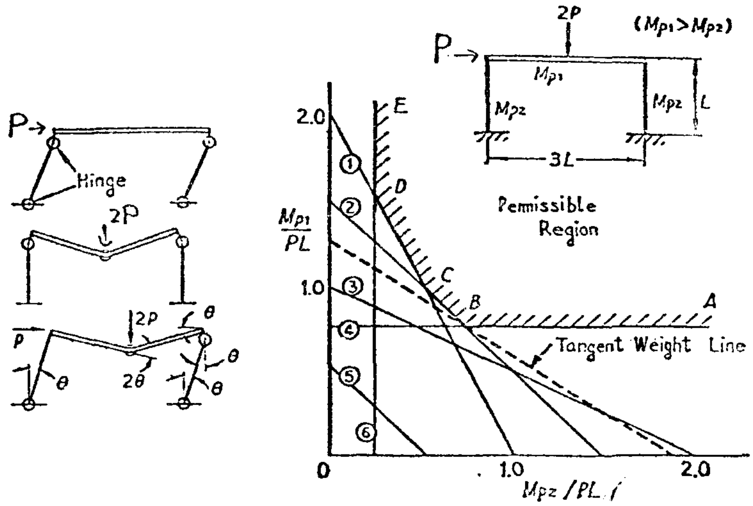


Fig. 8.

In order to minimize W_t, it is necessary to minimize the following weight function W_f,

$$W_f=\sum M_{pi}L_i \tag{5}$$

Therefore, it is shown that weight of the frame is proportional to the distance between the origin and this straight line. (5). Then you may easily find in Fig. 8 where the optimum point is.

3. Buckling

3-1 Therminoloay

Buckling (○) { Elastic stability / Inelastic stability } Bifurcation Eigen value problem
Instability (×)——Maximum load

Buckling is stable phenomenon, and the load moves from stable region to stable region, on the contrary

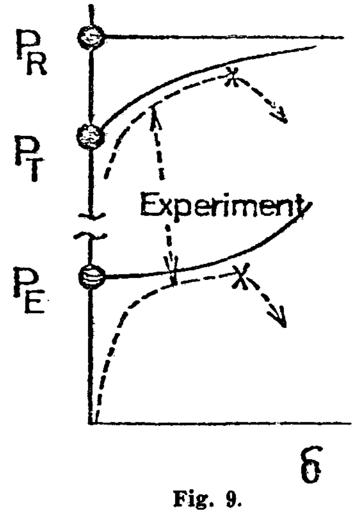


Fig. 9.

in case of instability, the load moves stable region to unstable region via the max. load point.

3-2 Experimental method of determining the critical buckling point of rectangolar plate subjected to inplane compression.

Many papers have been published on the instability of plates in the plastic range. However, it has been a difficult problem for researchers to determine the buckling load of a plate from experimental data. Several approaches have been used. The critical buckling load (or applied average strain) has been assumed to be equal to the maximum load (or the corresponding strain) carried by the plate. Another approach was to take the point of a rapid increase in deflection or a point of bifurcation of strains measured on both surfaces at a certain point on the plate.

Many researchers, such as Haaijer [5, 6] and Massonnet[7], have studied experimentally and theoretically the compressive strength of plates in the plastic range. However, the method of determining the critical load of the compressed plate in the plastic range had not been discussed theoretically. The critical bucking load of the specimen was determined principally by observation or as the maximum load. For example, Fig. 10 shows results obtained by Haaijer. The critial buckling strain was determined by observation as indicated in the figure.

The "$\varepsilon-\delta^2$ method" is introduced for determining the critical load from experimental data. A theoretical justification is furnished by using the energy approach and the deformation theory of plasticity and abstained the following equation.

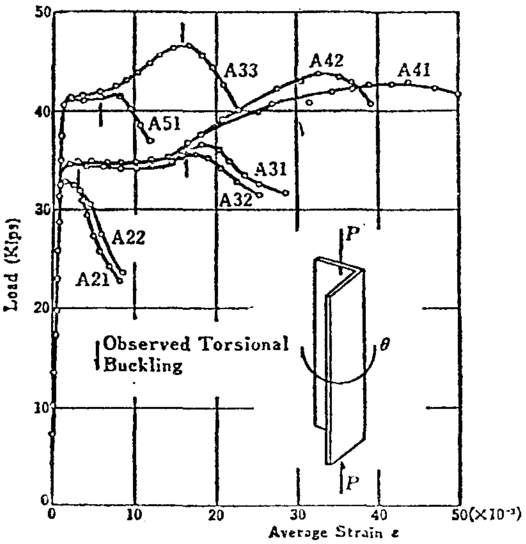


Fig. 10. Results of Angle Compression Tests (Haaijer and Thurlimann, Ref. 5)

where $$\varepsilon \simeq \varepsilon_{cr}+a_2\left(\frac{\delta}{a}\right)^2$$

$$a_2=\frac{L\varepsilon^2{}_{cr}+N}{3\varepsilon^2{}_{cr}-2K\varepsilon_{cr}-M}$$

This equation clearly indicates a liner relationship between ε and δ^2. L, M, N and K are contant for each boundary condition, for example, in case of fixed-fixed (loaded edges are simply supported), the assumed deflection is

$$\left.\begin{aligned}
&w=\frac{\delta}{2}\cos\frac{\pi x}{a}\left(1+\cos\frac{2\pi y}{b}\right)\\
&K=-\left(\frac{1}{\lambda}-1\right)\varepsilon_0+\frac{\pi^2}{9}\left\{\left(\frac{b}{a}\right)^2+\frac{16}{3}\left(\frac{a}{b}\right)^2+\frac{8}{3}\right\}\left(\frac{t}{b}\right)^2,\ L=\frac{35}{192}\pi^2\\
&M=\frac{\pi^2}{9}\left\{\frac{1}{4}\left(\frac{b}{a}\right)^2+\frac{16}{3}\left(\frac{a}{b}\right)^2+\frac{8}{3}\right\}\left(\frac{1}{\lambda}-1\right)\varepsilon_0\left(\frac{t}{b}\right)^2\\
&N=\frac{\pi^2}{108}\left\{\frac{35}{32}+10\left(\frac{a}{b}\right)^2+14\left(\frac{a}{b}\right)^4\right\}\left(\frac{1}{\lambda}-1\right)\varepsilon_0\left(\frac{t}{a}\right)^2
\end{aligned}\right\}$$

where $\lambda=Et/E$ $E=$Young's modulus

$Et=$tangent modulus

This method can be described briefly as follows. The average compressive strain of a plate, ϵ, is plotted against the square of the maximum deflection of the plate, δ^2. In this diagram, the critical strain ϵ_{cr} of the axially compressed plate may be easily and uniquely established by the intersection of the ϵ-axis with a mean straight line drawn through the plotted points. Critical loads determined experimentally by the method show a fair agreement with theoretical values. (See Fig. 11) This approach is also applicable in the elastic range of plate buckling.

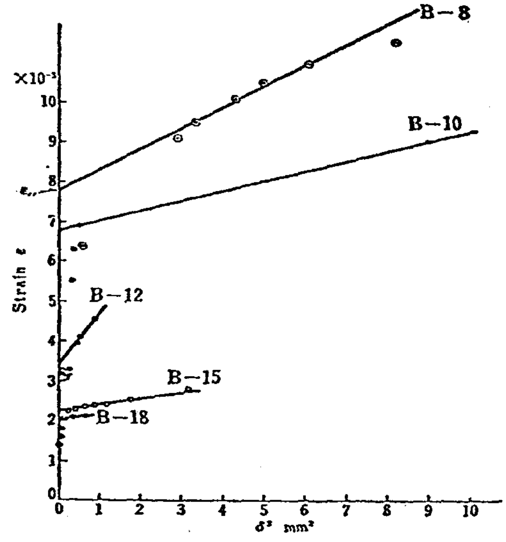


Fig. 11. Relationships of ϵ versus δ^2 for Specimens of B—Material.

3-3 Compressive strength of plate with a hole

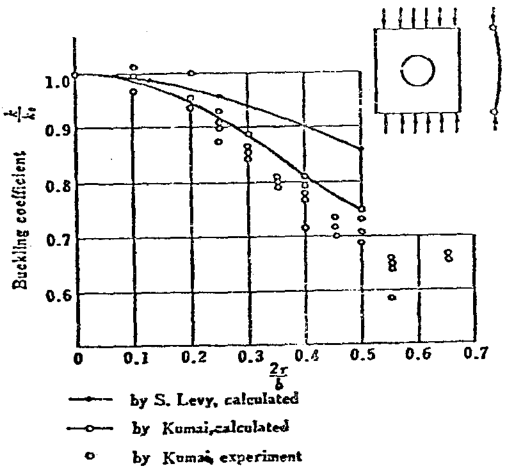


Fig. 12.

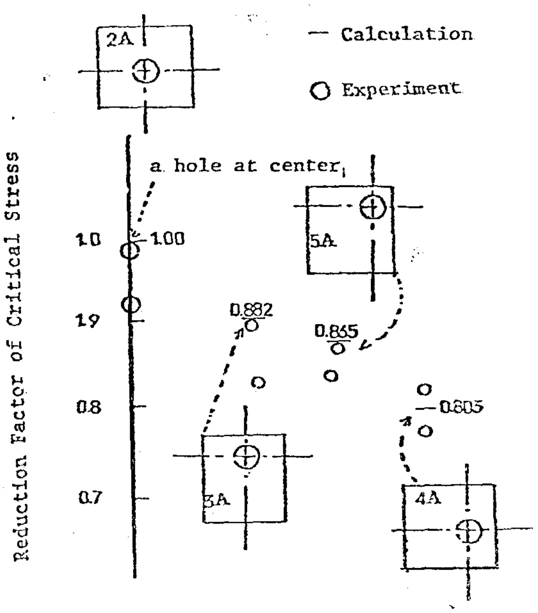


Fig. 13.

3-4 The Influence of residual stresses on the compressive strength of members

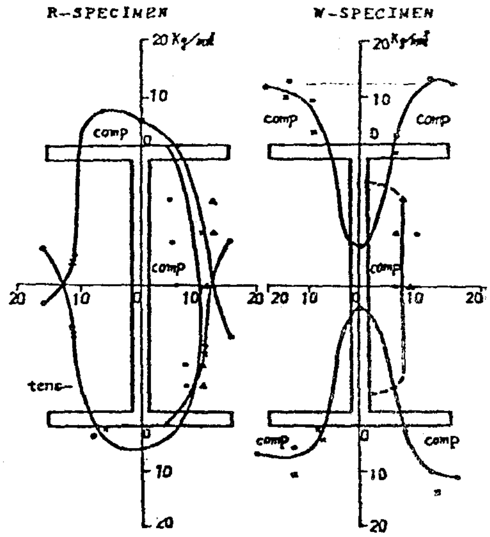
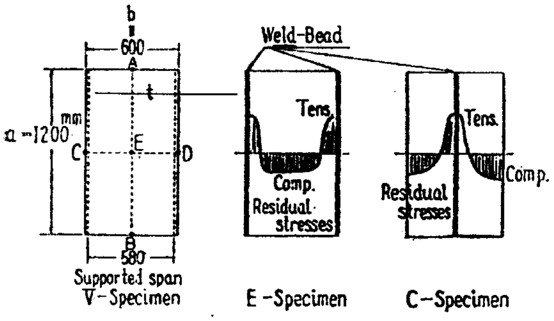


Fig. 14. Residual Stress Distribution

4. Recent Studies on Non-liner Response in Japan

Taking account of the past experience of catastorophes occured in rough seas near Japan, the main efforts of investigation have been devoted to the nonliner structural response of ship structures including the evaluation of their ultimate strength as well as elastic stress analysis for design purposes by using Finite Element Techniques. With regard to non-linear structural response, the emphasis has been put on three main themes as follows:

(1) To clarify the influence of initial imperfections on the load carrying capacity of structures for design



Specimens

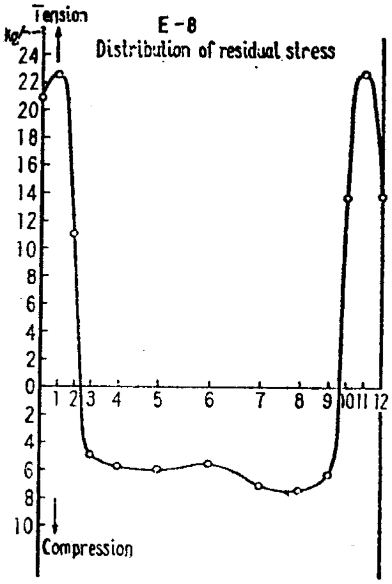


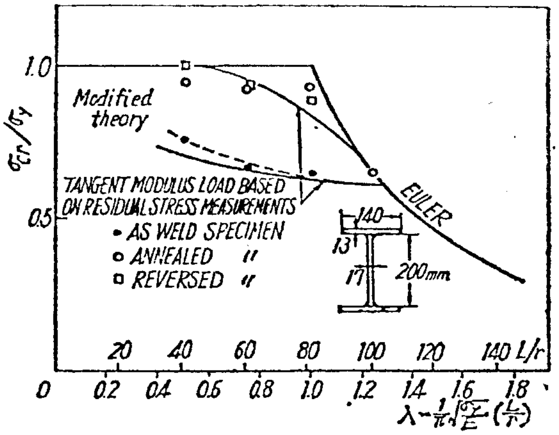
Fig. 15.

purposes, structural elements such as plates, columns, girders and others having initial imperfections, have been studied experimentally and theoretically.

(2) When the strength of rather large scale structures such as transring or large stiffened plate is calculated, simplification in computation of Finite Element Method has been tried by using the idealization and/or simplification of the behaviour of structural elements which compose the structure, in order to avoid huge amount of computing time and expenses.

(3) In addition to the above, a keen interest has been paid to dynamical response of structures induced by wave pressure and other impulsive loads.

In relation to the above-mentioned problems, Research Committee 127 [11] (abbreviated as SR 127)



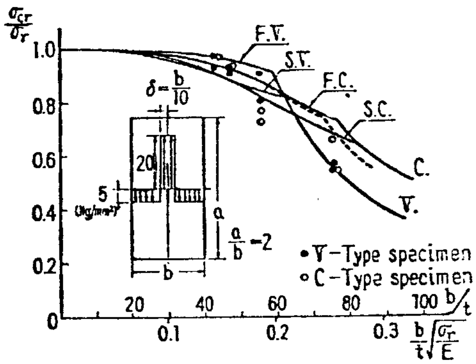


Fig. 16.

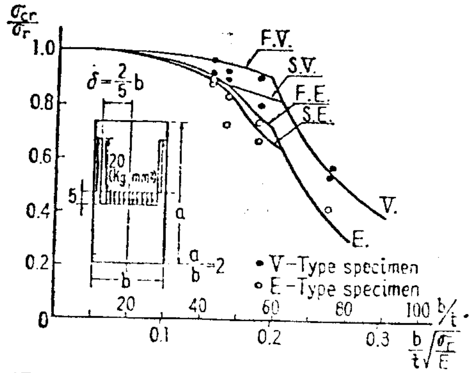


Fig. 17.

organized in the Shipbuilding Research Association of Japan treated the effect of initial deformations in hull elements on their strength and measured shapes and amounts of imperfections in actual ship structural elements such as stiffened panels, struts and so on. As an example, the effect of initial deformation on the strength of square plate is briefly introduced in the next article.

On the other hand, SR 133 Comm. [12] titled "Experimental Researches on Strength and Fracture of Ship Structures", mainly studied ore pressure in tanks of ore carriers and collapse of transverse ring of the carriers with a strut, and some of these results are illustrated at the end of this article.

Literature Concerned

Papers closely connected to ship structural strength and published in Japan recently, are hereafter briefly introduced.

With regard to the post-buckling behaviour of columns subjected to a concentrated load, columns with $1/r<50$ was studied by T. Kato, and in this [13] columns are tested to endorse the theory and it is found that strain hardening and length of yield level have much effect on the post-buckling behaviour of columns. Suzuki and Ono [14] carried out experimental studies on the relationship between the deformability of structures and properties of High Tensile Steels used, and found that the deformability of the beam subjected to bending, is proportional to $\left(\frac{1}{\sigma_y}\right)^2$. Saiso and Tanaka [15] calculated by using Finite Elements Techniques, the stiffness $(Ku)_{cr}$ of lateral supporting members to prevent lateral buckling when the beam member becomes fully plastic, and found that $(Ku)_{cr}/Ko=5$ for gradient moment and $(Ku)_{cr}/Ko=2$ for uniform moment, where $Ko=P/\delta$, P=horizontal load applied at the midspan of simply supported beam and δ=horizontal displacement at midspan.

Fujimoto et al. [16] treated the strength of columns on the basis of probablistic theory using Monte Carlo Method. The influence of random initial deflection, residual stresses and yield stresses on the buckling strength is as follows:

(1) Coefficient of variations of buckling loads becomes maximun near

$$\lambda=\frac{1}{\Pi}\sqrt{\frac{\sigma_y}{E}}\left(\frac{2}{r}\right)=1.0 \tag{1}$$

(2) When initial imperfections such as yield stress and initial deflections, exist simultaneously, it is shown that standard deviation of buckling loads in this case is approximately equal to root mean square of each standard deviation where only one kind of imperfection exists.

Okatsu [17] studied the effect of biaxial imperfection on column strength, and tested Hcolumns with initial deflection. It is clarified that numerical elastic-plastic calculation shows a fair agreement with experimental results.

M. Wakabayashi et al. [18] conduct an experimental investigation on the elastic-plastic behaviour of rahmen structures. When the framed structure is subjected to horizontal load, slenderness ratio of columns and rigidity ratio of members become main factors affecting the behaviour of the structure. The following items are shown from the experiment:

(1) When the same axial force is subjected to rahmen structure, horzontal strengh decreases as members become slender.

(2) As the decrease of axial force, the peak load appears in the load-deflection curve. Critical axial force at which the peak load appears is related to slenderness ratio and rigidity ratio, and generally speaking it becomes larger as the slenderness ratio decreases and the rigidity ratio increases.

(3) Calculated load-deflection curve by using plastic hinge method is different from the experimental one. However, regarding to the horizontal strengh, calculated values show a good agreement with test results in case of framed structure with slender members.

J. Suhara et al. [19] present a method of analysing the strengh of framed structures by means of using elasto-plastic hinge method in which the effect of axial force is taken into account. Using this method, it is not necessary to cheak all the possibe collapse mechanism. Collapse load and mechanism can be automatically obtained by pursuing the elasto-plastic hinges one by one. Therefore this method becomes very useful for the determination of the collapse load complicated many stories framed structures.

Structural elements usually have several kinds of imperfections such as initial deflections, residual stresses and so on, and it is known that these imperfections may sometimes weaken the strength of structures. As a fundamental problem Y. Fujita et al. [20] investigate rectangular and H type cross section columns in order to know compressive strength of columns with initial imperfections. As to load-deflection curve and maximum load, calculated values by using finite element technique show a fairly good agreement with experimental ones. In case of rectangular cross section columns with sine wave imperfection, it is shown that columm curves agree practically well with the values obtained by Jezek.

S. Ando et al. [21] have aso developed a computer program along the same line abovementioned. As examples, caculations are made on columns subjected to eccentric compressive load and on columns with initial deflection.

T. Usami [22] analyses the post yielding behaviour of thin walled open section member subjected to axial force, bending moment and uniform torsional moment, and obtains satisfactory results comparing with experimental ones. It is thown that calculated forces of the member show a good agreement with the upper bound solutions obtained by Gaydon.

Using the finite element technique, H. Kojima et al. [23] analyse framed structure in which the decrease in rigidity due to local buckling of members is taken into consideration. Load incremental method is applied; that is when a member forms a plastic hinge or buckles, end conditions of the member are revised and calculation is continued till the mechanism collapse or unstable collapse due to buckling is attained. The conclusion is that: When the final collapse due to elastic buckling, the obtained collapse load coincides with the ordinary buckling load. On the other hand, when it does due to mechanism collapse, rather lower strength is obtained compared with the strength caclulated by using ordinary elasto-plastic analysis technique.

In order to save computing time, Y. Maeda et al. [24] introduce acceleration approach on analysis method for framed structure. New incremental deformation is estimated by the total deformation produced up to the last step and new epuilibrium equation is constituted by using the obtained incremental deformation. As examples, truss, arch and curved beam are calculated.

Using the finite difference method, Y. Aoshima et al. [25] formulate an analytical technique to treat thin walled open section column subjected to biaxial bending and axial force. They studied more generlaized boundary conditions for columns, considering the fact that stiffness of a cloumn which is a structural member of a rahmen structure is largely affected by the stiffness of beams connected to the column at the ends.

Structural members are often subjected to various kinds of heat treatment such as welding, gas-cutting and quenching, etc. during the process of fabrication. Concerning thermal stress analysis, Y. Ueda et al. [26] have developed a basic theory for thermal elas-

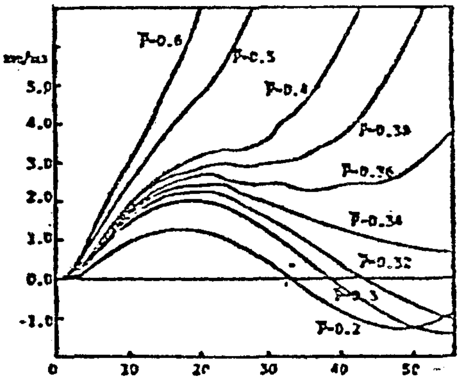


Fig. 18. Deflection rate of column at midspan

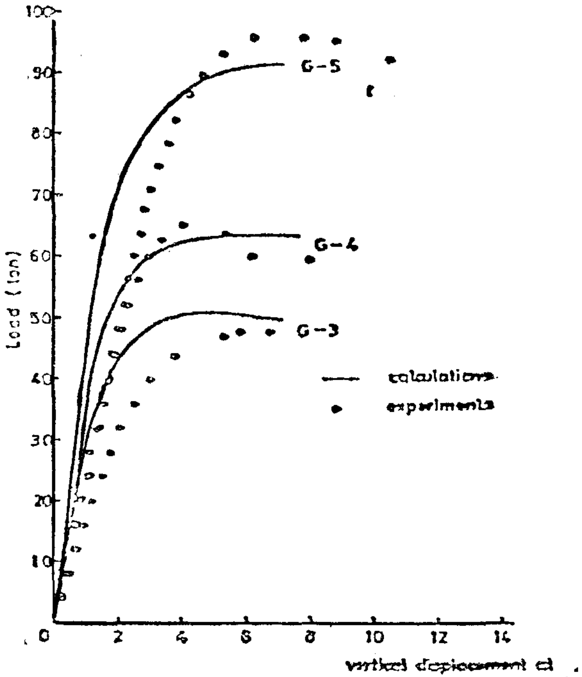


Fig. 19.

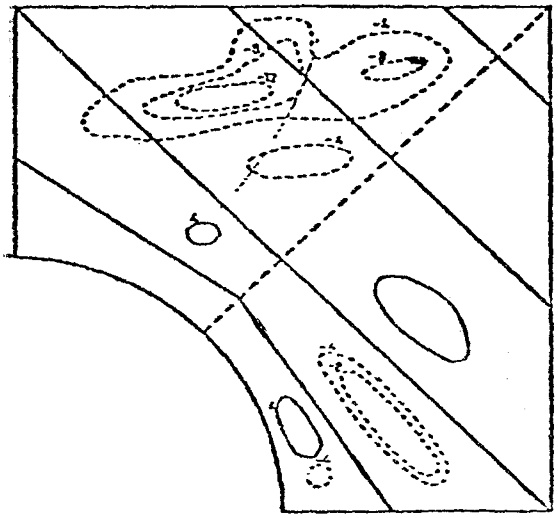


Fig. 20. Calculated additive out-of-plane deflection pattern of corner part of G-5 at the collapse load (91.2ton)

tic-plastic analysis based on the finite element method with the incremental procedure. considering the influence of temperature on the materal properties, As an example, non-liner behaviour of a beam subjected to axial force and temperatre change is traced.

Non-linear dynamic problems such as dynamic problems such as dynamic column strength and dynamic tension of notched plate, are treated by Ueda et al. [27] by using Finite Element Technique. Especially in case of dynamic elatic-pslastic buckling of columns with initial deflection ($\delta_c = \frac{1}{4}$depth of column), an interesting example of calculation had been obtained (see Fig. 18). In this example,

P_{cr} dynamic $\simeq 0.36P$

P_{cr} static $\simeq 0.42P$

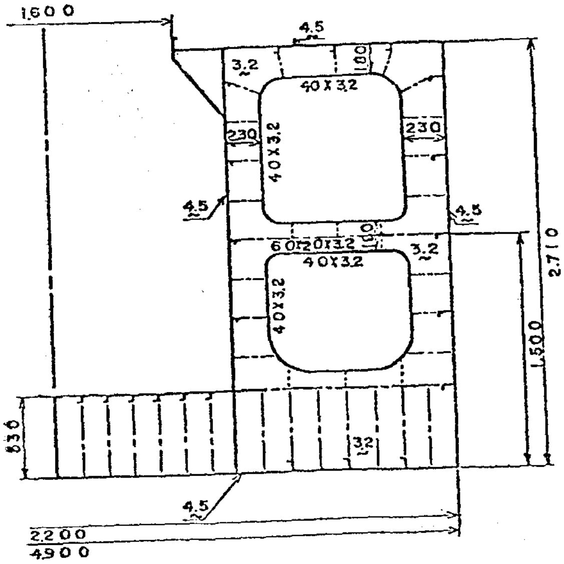
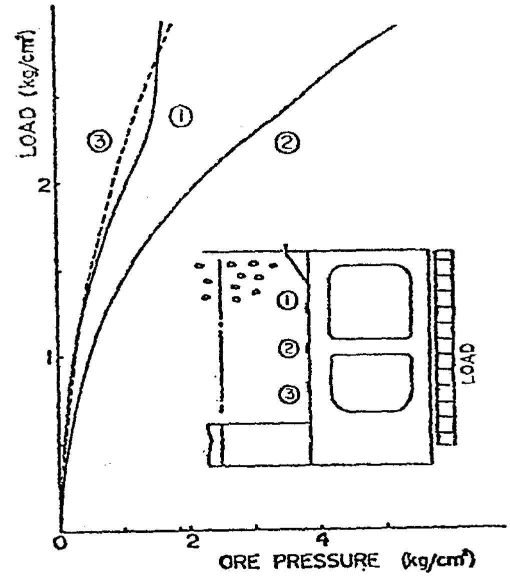
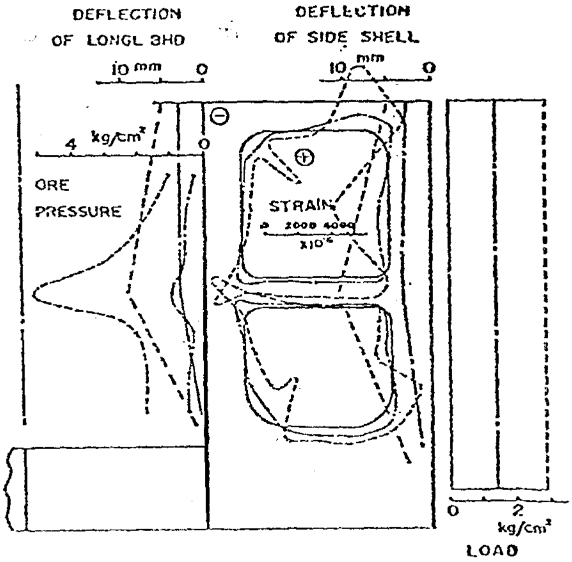


Fig. 21.





With regard to the ultimate strength of girders, Hasegawa et al. [28], [29], [30] studied the adequate web-stiffening method as a problem of the elastic-plastic buckling of stiffened panel subjected to bending and thrust, by using the finite strip method.

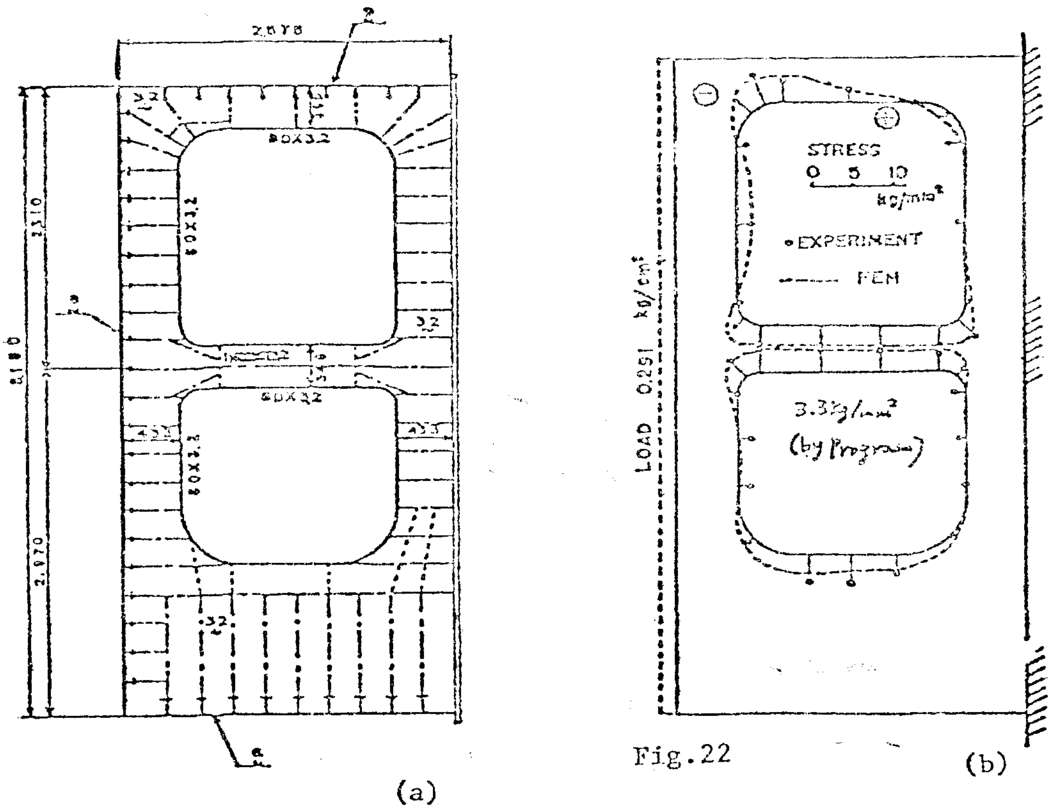
Fujita et al. [31] carried out the collapse tests of three structural models with initial imperfection, and compared these results with numerical calculation. Calculated maximum loads and deflections show a fair agreement with the computation by F. E.M. (see Fig. 19, 20)

To investigate the effects of initial imperfections on the strength of square plate, simply supported 500×500^{mm} square plates had been tested by Ueda et al. [32], and it is concluded as expected that the larger the initial imperfection and plate thickness are, the less the load carrying capacity of the plate is.

Ueda and Rashed [33] developed and anlytical unit of large size and idealization of the highly non-linear to overcome enormous computing time in case of analysing more complicated structures such as ship structures by using Finite Element Techniques. In this analysis, a "Girder-Element" is proposed and this method promises the possibility of computing the ultimate strength of structures of large size.

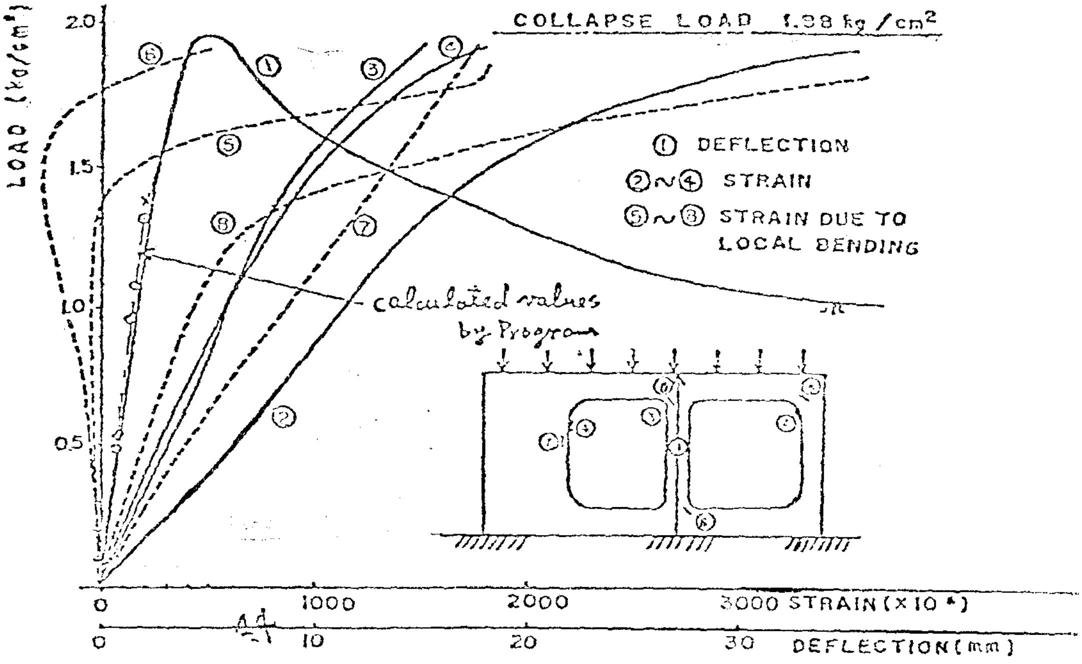
H. Arai [34] presents a finite element analysis on the non-linear problems of plate structures. Triangular plate element with three subelements is used, and beam element is also used for stiffening members of the structures. As an example of the theoretical analysis, numerical calculations are made on the following models, such as, initially deformed square plates, square plates with a circular hole, girder with round corner and so on. Comparison is made between theoretical predictions and test results on the specimens. It is concluded that the present method can be generally applied to the analysis of elasto-plastic behaviour of arbitrary shaped plate members for the purpose of estimating the buckling, yielding and ultimating strength of the plate structures.

As a part of committee works abovementioned, Y. Yamamoto et al. [35] investigated dynamic collapsing



(a) (b)

Fig. 22.



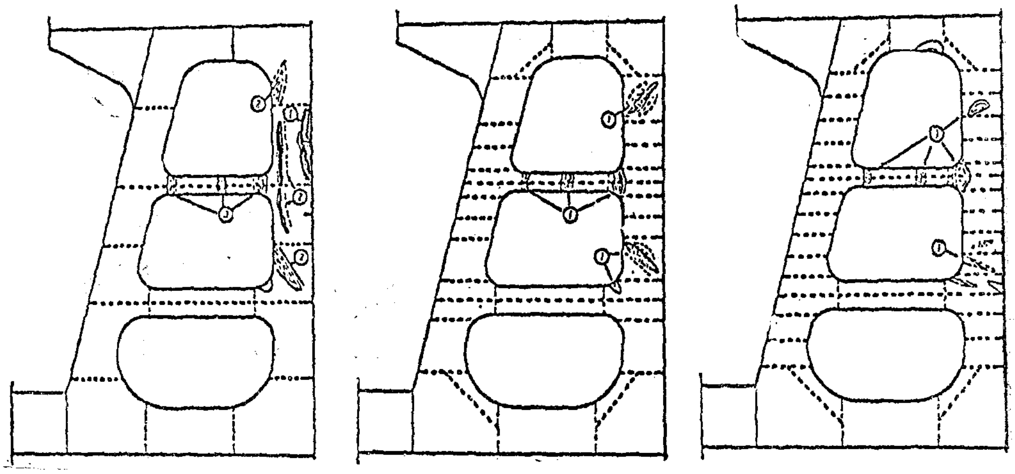


Fig. 23.

of side shell structure of an ore carrier under non-impulsive wave pressure, and clarified that the amount of deflection of the side shell caused by dynamical collapse of strut due to quasi-static wave load is almost two times as much as that caculated by statical analysisis.

Some results of experimental works carried out by SR133 Comm., are illustrated in the following;

5. The Influence of Initial Imperfection on the Compressive Strength of Plates

5-1 Allowable limit of imperfection

As you may know, in Japan naval architects have been studying the influence of imperfection on the strength of structural members since 1966, especially the 127th Research Committee (SR 127) [11] as described above, had carried out extensive studies on the effect of initial imperfection. The aim of this Comm. is to extablish the allowable limit of tolerence in fabrication of ship structures, for example the allowable limit of initial deformation of flat panel (See Fig. 26), and then to achieve more rational and economical design as follows:

(1) In the stage of fabrication, every imperfection is quality controlled less than the above-mention allowable limit.

(2) To establish the method to evaluate the influence of imperfectiion on the strength of structural members.

(3) To reduce the factor of safety (or load factor) by an amount resulted from the expected imperfection at the early strage of structural design, because the reduction of strength due to imperfection had been thought to be included in the factor of safety as an ignorance factor.

There are three ways of establishing the standards of acceptable or allowable limit of initial imperfections in structures. As an example, the case of JSQS[*1][36] is shown in the following scheme. (Fig. 24)

(1) Simple one may, obviously, be like JSQS which bases on the data of imperfection measured in actual ship's hull. In case of JSQS, as known, there are "standard range" and "tolerance limits", and the standard range occupies 95% in probability and only less 0.3% is beyond tolerance limits, refering to ships ever built in Japanese shipyards. That is, this standard fully depends only on the present practiced technical level of quality control in shipyards.

(2) The allowable tolerance may deterministically be determined by the capability of a member whih has initial imperfections. For example, the limit may be selected in such a way that ultimate strength of the member should be more than, say, 90% of that of an ideal member.

(3) More reasonable way to define the allowable tolerance limit may be based on probablistic theory, i.e., in any case, probability of failure of a member under properly adopted load conditions, is maintained above a certain level required according to the

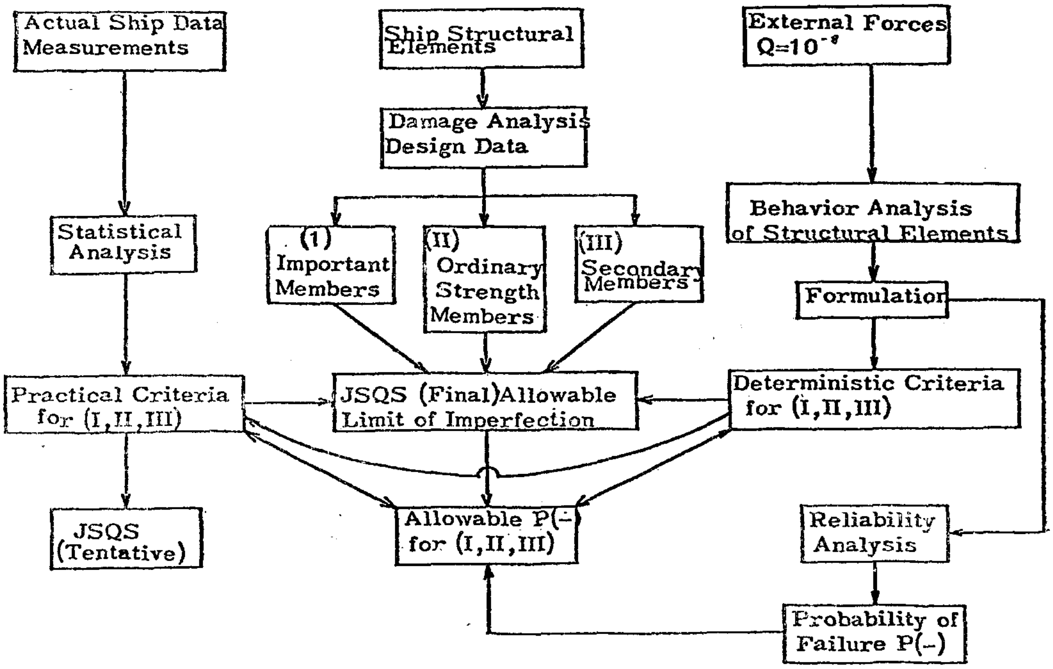


Fig. 24. Scheme for Allowable Limit of Imperfection of Ship Structural Elements

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[*1] JSQS meams Japanese Shipubilding Quality Standard.

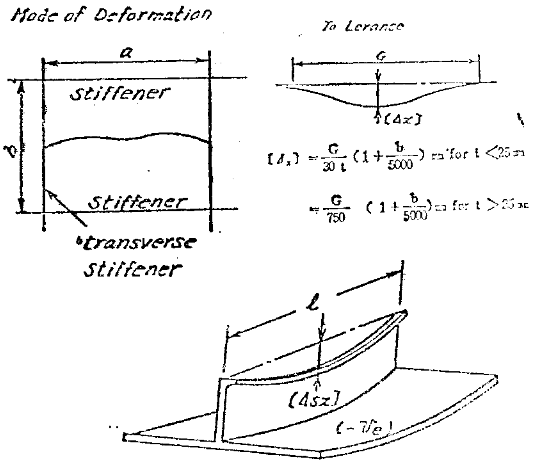


Fig 25. Examples of fabrication tolerances

grade member.

It is known that keen attention has been paid to the effect of initial imperfections (such as initial deformation as well as residual stresses) on the structural capability of members, since the disastrous accidents of long span box girder bridges such as Der Neuen Danau Bridge (1969), Milford Haven Bridge (1970), West Gate Bridge (1970), Koblenzer Südbrücke (1971) and so on had failed due to buckling or collapse of main panel under construction. Especially in U.K. committee of investigation extensively into the design and erection of steel box girder bridges (called Merrison Committee) investigated the case of Milford Haven Bridge [37] and reported that bridge design should be compatible with unservicability design and collapse design and also showed the

Fig. 26. Unfairness in ship structures (JSQS, 1975)

Division		Deformation		UNIT: mm	
Section	Sub-section	Item	Standar Range	Tolerance limits	Remarks
Unfairness	Shall plate	Parallel part side	4	6	
		Parallel part bottom	4	6	
		Fore and aft part	5	7	
	Double bottom tank top plate		4	6	
	Bulkhead	Longl Bulk head Trans Bulk head Swash Bulk head	6	8	
	Strength deck	Parallel part (Between 0.6L)	4	6	
		Fore and aft part	6	9	
		Covered part	7	9	
	Second deck	Bare part	6	8	
		Covered part	7	9	
	Fore-castle deck Poop deck	Bare part	4	6	
		Covered part	7	9	
	Super Structure deck	Bare part	4	6	
		Covered part	7	9	
	Cross deck		5	7	
	House wall	Out side wall	4	6	
		Inside wall	4	6	
		Covered part	7	9	
	Interior member	Web of girder, trans	5	7	
	Floor and girder of double bottom		6	8	

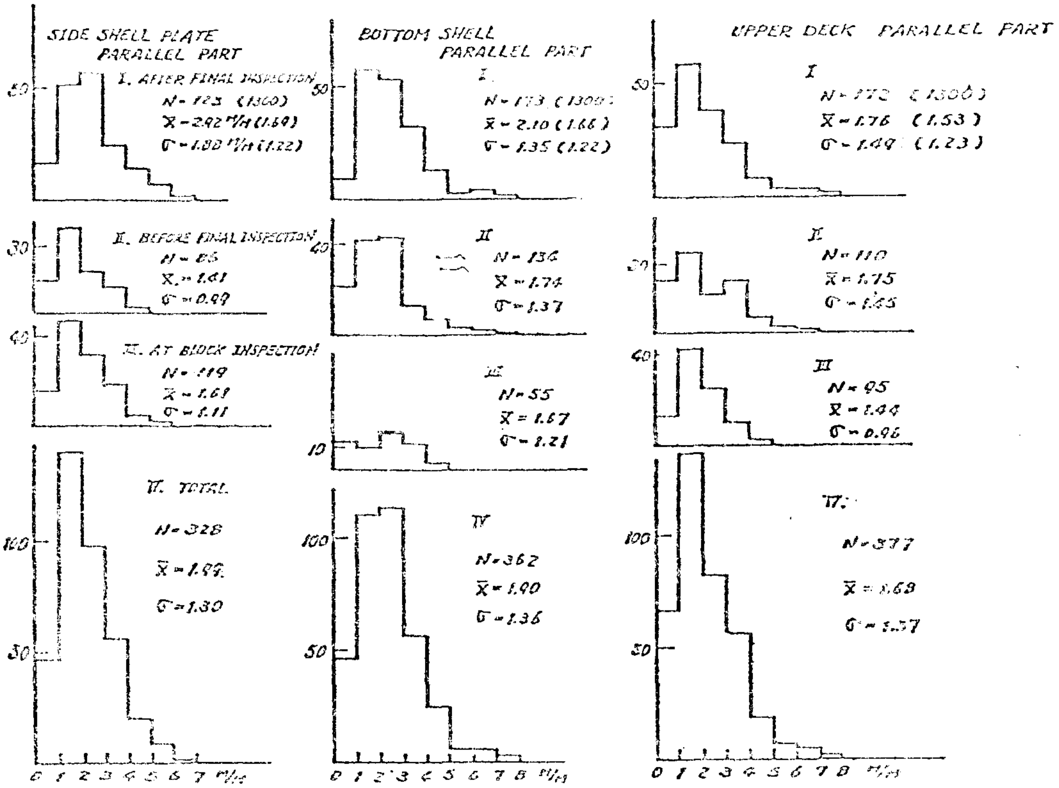


Fig. 27. Distribution of unfairness (Number in referred to data of J.S.Q.S. 1971)

tolerances of structural element shown in Fig. 25,

But in no case less than 1.0mm in flange and diaphram panels and in unrestrained web panels in compression and not less than 3.0mm in other web panels.

$$[\Delta_{sx}] = -\frac{l}{1200}$$

or 2.0mm, whichever is the greater

5-2 Japanese Shipbuilding Quality Standard (JSQS)

As described above, the first edition of this standard had been published in 1966 and then up to the present several revisions have been done. Originally, the standard was a sort of guidance for quality control in shipyards, however, since then many theoretical studies have been carried out, e.g., the research works of the SR127 Comm. [11] etc. to provide theoetical background to the standard, so it may be said that the present JSQS has become a real standard for Japanese shipbuilding techniques. Of course, these kinds of standards should be incessantly tested for their availability with ever changing technical innovationand be subjected to additions and alterations in future along withthe development of shipbuilding practice. Fig. 26 shows an example of JSQS. As a reference, examples of actual measured results (carried out by the SR127 Comm.) are shown in Fig. 27.

5-3 Studies on the Influence of Initial Imperfeccion on Compressive Strength of Stiffened and Unstiffened Plates

Recent theoretical and experimental studies [38], [39], [40], [41], [42] have led to a much improved understanding of the influence of residual stress and initial deformation of strength and stiffness of square and rectangular plates under uniaxial compression. Fig. 28, 29, Some illustrative results, computed for square plates with unloaded edges constrained to remain straight [42], are shown in Figures 28 to 29. These results are in the form of load-shortening curves ($\sigma'_x=\sigma_{ave}/\sigma_y$ plotted against $\epsilon'_x=\epsilon_{ave}/\epsilon_y$) whose slopes provide an indication of effective plate stiffness $\frac{d\sigma_{ave}}{d\epsilon}$ up to and beyond collapse; the curves have

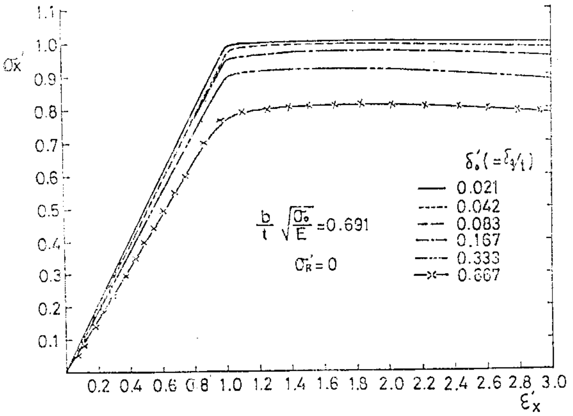


Fig. 28. Average stress-strain curves for plate in compression with constrained edges(ref. [8]) effect of varying initial deformation

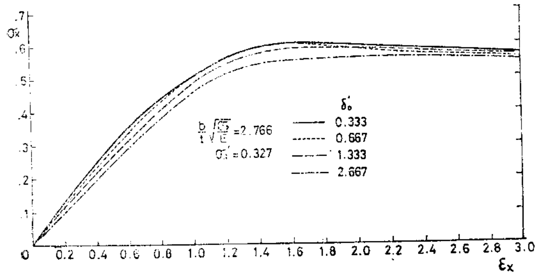
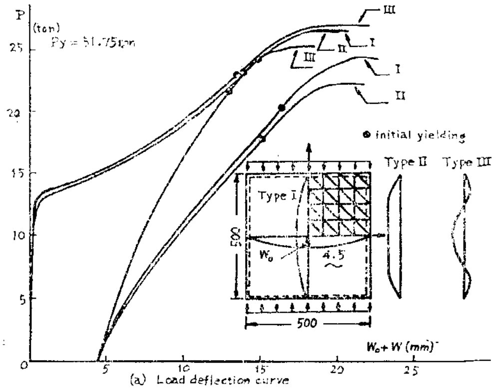


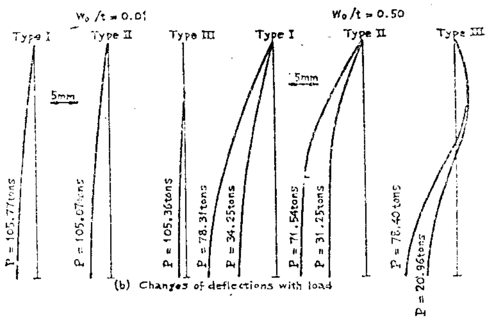
Fig. 29. Average stress-strain curves for plate in compression with constrained edges(ref. [8]) effect of varying residual stress

been computed for a range of plate slenderness$\left(\frac{b}{t}\right)$, plating distortion (asumed to be doubly sinusoidal with amplitude δ_0)and idealized weld-induced compressive residual stress σRC (non-dimensionalized in Figures 5 to 6 as $\sigma' R=\sigma RC$/yield stress). It is clear from these results that residual stress and initial distortion may both exert an important influence on plate stiffness and strength. Theoretical analysis [38] has shown that the form of initial deformation most strongly influencing plate strength is the "ripple" component having a half-wavelength, approximately equal to the width b of the plate. Overall distortion, involving a single half-wave over the plate length, has comparatively little effect on strength under longitudinal compression. Overall distortions may however strongly influence the strength of plate panels subjected to transverse or biaxial compression.

M. Kmiecik[43], [44], and T. Borzecki[45] studied the effect of initial deformation on the compressive

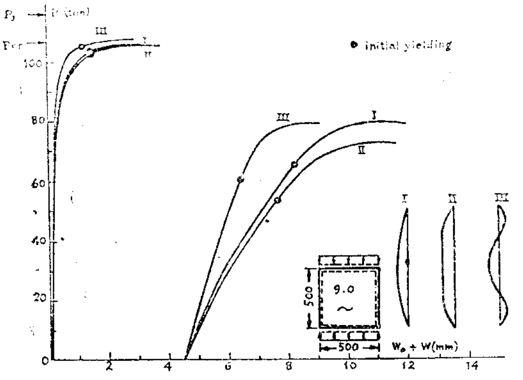


(a) Load deflection curve

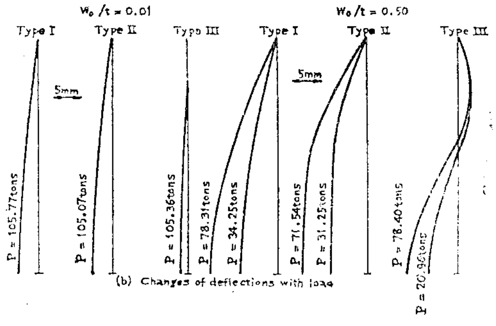


(b) Changes of deflections with load

Fig. 30. Effect of the shape of initial deflection (t=4.5mm)



(a) Load deflection curves



(b) Changes of deflections with load

Fig. 31. Effect of the shape of initial deflection (t=9.0mm)

strength of stiffened plates and stated that two factors, the magnitude of the initial deformations and its shape should simultaneously be taken into account when considered the effect of initial deformation on strength of axially compressed plates, the latter playing in much greater role than the former.

Y. Ueda et al. [11] studied "Ultimate Strength of Square Plate with Initial Deflection and Residual Stress Subjected to Compression" as a part of SR127 Comm. works. As a fundamental study, the regidity and strength of square plates under compression are analysed by the finite element method, and series of experiments are conduced. The effect of the shape of initial deflection, the effect of the magnitude of initial deflection, and then the influence of welding residual stresses with initial deflection are treated. Elastic-plastic large deflection analysis is carried out for 500×500mm square plates, of which thicknesses are 4.5mm and 9.0mm. Three types of initial deflection are assumed, and the ratio of the maximum deflection to the plate thickness, w_0/t are chosen as 0.01 and 1.00 for t=4.5mm and 0.01 and 0.50 for t=9.0mm, respectively. The plates are assumed to be simply supported along four edges, and the loads are applied so as to give constant displacements on loading edges. The load-deflection curves and the changes of deflection shape are shown for t=4.5mm and t=9.0mm in Fig. 30 and 31, respectively. The finite element representation and the types of initial deflection are also shown in Fig. 30. In both cases, t=4.5mm and t=9.0mm, the difference of the shapes of initial deflection have a very little influence on the rigidity and the ultimate strength in the case of small w_0/t. However, when w_0/t is large, there exists some differences in the behaviour of the plates, such as the rigidity and the ultimate strength.

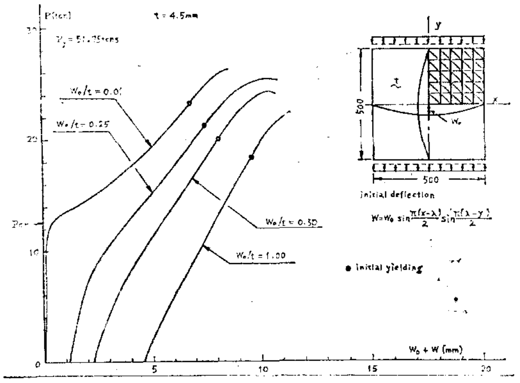


Fig. 32. Effect of the magnitude of initial deflection (t=4.5mm)

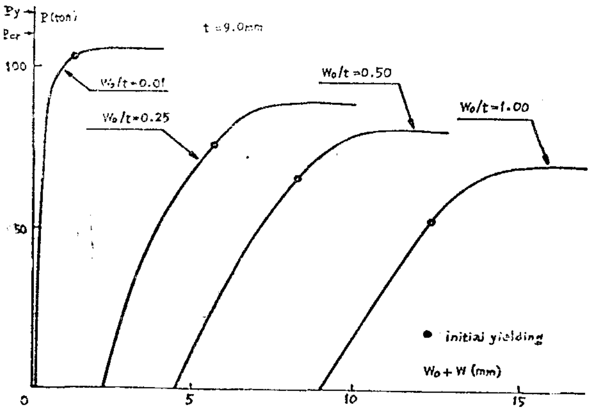


Fig. 33. Effect of the magnitude of initial deflection (t=9.0mm)

500×500mm square plates of which thicknesses are 4.5mm, 9.0mm and 12.7mm are analysed. The shape of the initial deflection is of a sine wave, and the ratios of the maximum deflection to the plate thickness are chosen as 0.01, 0.25, 0.50 and 1.00. The load-deflection curves for each plate thickness against each w_0/t are shown in Fig. 32, 33 and 34. The finite element representation and the initial deflection are also shown in Fig. 32. In each case, the rigidity and the ultimate strength become lower as w_0/t becomes larger. The decrease of the ultimate strength is more apparent as the plate becomes thicker. Influence of

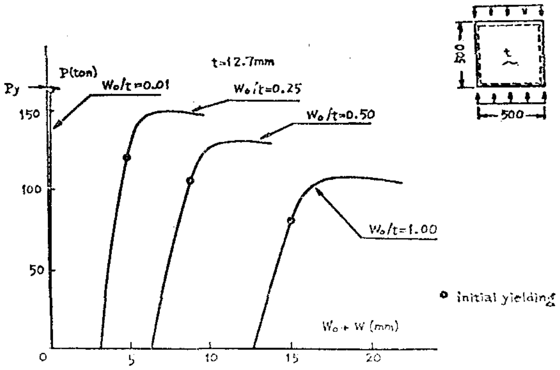


Fig. 34. Effect of the magnitude of initial deflection (t=12.7mm)

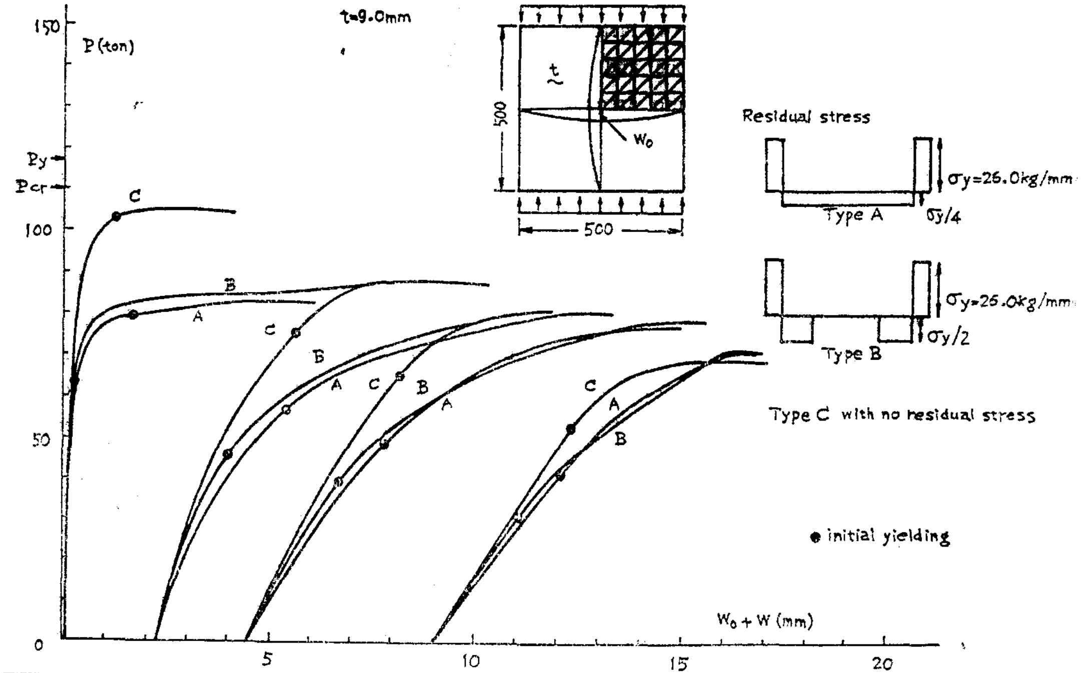


Fig. 35. Effects of the magnitudes of initial deflection and the residual stresses (t=9.0mm)

the residual stresses 500×500mm square plates of which thickness is 9.0mm are also analysed. Two types of residual stresses are assumed only in one direction as shown in Fig. 35. The residual stresses of type A and type B are usually observed in thick plates and thin plates respectively. The load-deflection curves are also shown in Fig. 35. For small w_0/t, remarked decrease of the ultimate strengh due to the residual stresses is observed. However, when w_0/t becomes large, this decrease is not appreciable. A series of experiments is carried out to examine the influence of the initial deflection and residual stresses, and the results are shown in Fig. 36, 37 and 38 for each plate thickness. The calculated curves are a little higher than the experimental results for t=4.5mm and 9.0 mm in the case of without residual stresses. However, taking into accout of the sensible character of the behaviour of the plates, both results are fairly well for all cases when only initial deflection is concerned. The relation of the ultimate strength against the thickness to breadth ratio, t/b is shown in Fig. 39.

Theoretical analysis of inelastic flexural buckling of

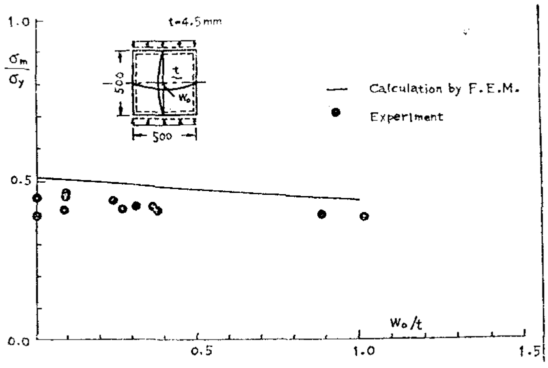


Fig. 36. Ultimate strength of 4.5mm thickness plates subjected to compression

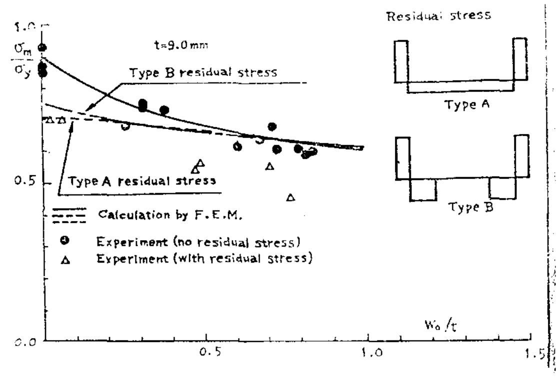


Fig. 37. Ultimate strength of 9.0mm thickness plates subjected to compression

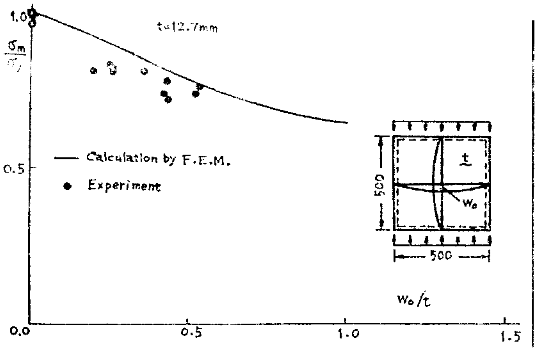


Fig. 38. Ultimate strength of 12.7mm thickness plates subjected to compression

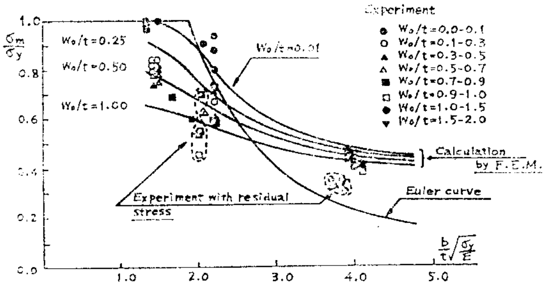


Fig. 39. Ultimate strength of square plates subjected to compression

stiffened panels, including approximate allowance for less of plate stiffness caused by buckling or yielding of the plating, has shown that the effects of initial stiffener deformations may be substantial and are very variable [46], [47]. The direction of initial deformation is important since the collapse strength of a stiffened panel is usually much lower where failure occurs towards the plating (with compression of stiffener outer-fibres augmented by bending) than where failure occurs towards the stiffener. It has been found that collapse of a "single-span" stiffened panel is comparatively insensitive to the amplitude of initial distortion towards the stiffeners but is much more sensitive to amplitude of distortion towards the plating [46]. In continuous or "double-span" systems important interactions, affected by the form of initial distortion, can occur between adjacent spans. Even where all initial distortions are in a favourable direction (towards the stiffeners) interaction between adjacent spans can cause failure to occur in the opposite, unfavourable direction.

The most important effect of compressive residual stress in a compressed plate is to cause premature yielding which reduces the axial stiffness of the plate (or of the plate-stiffener combination); in the case of a stiffened panel, extra load is then thrown into the stiffener leading to premature yielding of the stiffener and hence column failure of the stiffened panel at an average stress which may be well below σ_y. The destabilizing action of compressive residual stress may also significantly accelerate buckling of slender plates, in which residual stresses are normally small but in which elastic buckling stresses may also be very low.

As illustrates in Figures 28 & 29, a consequence of residual stress and associated premature yield in a compressed plate is that the level of compressive strain at which the plate's maximum load occurs is increased to as $2\ \varepsilon_y$. Except in the case of hybrid structures containing plating whose yield strength is higher than that of the stiffeners, this effect will tend to prevent collapse of a stiffened panel by "plate failure", i.e. collase in which the load carried by the plating reaches its maximum and then reduces rapidly before yield and hence buckling of the stiffeners occur. Collapse is therefore normally likely to occur by column-like buckling of the stiffened panel involving flexural and/or lateral deformation of the stiffeners; such failure will however often be very strongly influenced by reduction of plating stiffness.

Residual stresses in stiffeners can also strongly affect compressive strength by accelerating or delaying yield in sensitive regions. Compressive stresses in stiffener outer-fibres may be beneficial [46], [47]. The possibility arises of deliberately inducing favourable residual stress distributions in stiffeners by heat-treatment or cold-bending. It has been shown [47] that unfavourable cold-bending of stiffeners resulting in compressive outer-fibre residual stress can reduce the column strength of a stiffened panel by as much as 35%; similar strength reductions may occur in ring-stiffened cylinders with cold-bent frames under external pressure loading [47].

5-4 Statistical Approach to the Acceptable Limit of Imperfections on the Ultimate Strength of Structural Elements

Along with the (3) category described with regard to Fig. 24, the most significant work has been reported in the Report of the SR127 Comm. [11] of Shipbuilding Research Association of Japan. In this report, as a typical example, the ultimate strength of simply supported square plates subjected to compressive load, is studied so as to evaluate the allowable initial deformation of deck plating of tankers on the basis of probablistic analysis and actual data.

In this analysis, σ_y is taken as 24kg/mm^2 for mild steel and 32kg/mm^2 for HT 50 assuming that σ_y is a deterministic quantity and ultimate strength of plates is formulated as follows;

$$\sigma_u=\sigma_y\left(\frac{\beta}{\Delta_0+\alpha}+\epsilon\right)$$

where $\Delta_0=\delta_0/t$ (initial deflection/plate thickness)

$\alpha=0.5130+0.01225\ S/t$ for $\sigma_y=24\text{kg/mm}^2$

$=0.1530+0.01414\ S/t$ for $\sigma_y=32\text{kg/mm}^2$

$\beta=0.6868,\quad \epsilon=0.2846$

Under the assumption that the strength of plates and stresses induced by external forces, are normally distributed, the probability of failure of the plate is,

$$P_f=\frac{1}{\sqrt{2\pi}}\int_{\gamma}^{\infty}e^{-\frac{x^2}{2}}dx$$

where $\gamma=(\bar{\sigma}_u-\bar{\sigma}_T)/\sqrt{S^2\sigma_u+S^2\sigma_T}$

$\bar{\sigma}_u$=mean of σ_u

$\bar{\sigma}_r$=mean ofσ_r

$S^2\sigma_u$=varianceof strength

$S^2\sigma_T$=variance of stresses due to external forces

An example is shown in the Fig.40 in which the allowable δ_0/t is determined in such a way that the probability of failure of larger tankers beyond 100,000 DWT in which deck plate is composed of HT 50, shall be equal to that of smaller tankers (≦100,000 DWT) having mild steel deck plating.

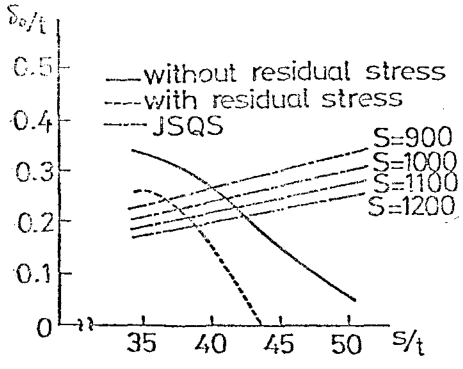


Fig. 40. Allowable limit of initial imperfection (δ_0/t v.s. S/t)

The above-mentioned example is only one of the ways to realize a numerical criterion based on probablistic way of thinking. However, it should be noted that there being many problems to be solved to establish this method and to enforce such standards to use in actual design, even though application of probablistic concept may by the most fruitful and powerful tool in this field.

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