ULTIMATE STRENGTH ANALYSIS OF ALL STEEL SANDWICH PANELS

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ABSTRACT

The research related to ultimate strength of all steel sandwich panels at the Ship Laboratory of HUT is reviewed in this paper. The studies include laboratory strength testing, numerical FEM analysis and development of design formulations for these panels. The ultimate strength is analysed under hydrostatic loading and under local point loading. Three cases can be classified for the collapse modes. For large loading areas and for small core plate thicknesses, elastic buckling of the core plate is the dominating collapse mode. For thicker plates, core yielding and buckling are causing the failure. The third type of collapse mode occurs when the face plate is thin, then the applied ultimate load causes high compressive bending stresses on the face plate causing face plate buckling before the collapse of the core plate.

INTRODUCTION

The demand for bigger, faster and lighter moving vehicles, such as ships and trains has increased the demand for efficient structural arrangements. Sandwich constructions offer one possibility for efficient utilisation of material. The present intrest in steel sandwich structures has been awakened by the developments in laser welding technology enabling efficient production of these panels.

The studies related to the all steel sandwich panels were initiated at HUT/Ship Laboratory in 1988, when the application of all steel sandwich panels as shell structures of an icebreaker was analysed (Tuhkuri, 1991, Tuhkuri, 1993). The applications on the shell of an icebreaker was found to be problematic due to high demand for the local strength of structures under ice loading.

Thereafter, the applications of all steel sandwich panels as deck and bulkhead structures of a cruising ship were studied (Kujala et al., 1995, Kujala and Tuhkuri, 1995). The studies include development of design methods, weight optimisation, ultimate strength testing under hydrostatic loading, and fire and noise testing of the panels. The steel sandwich deck and bulkhead structures were found to be 30 to 50 % lighter than the conventional steel grillages. These studies were continued by conducting fatigue testing of the all steel sandwich panels used as longitudinal bulkheads (Kujala and Kotisalo, 1996, Kujala and Salminen, 1997). Also the application of all steel sandwich panels as crane structures is studied by conducting fatigue testing for laser welded beams (Kujala et al., 1996).

The design methods for all steel sandwich panels were further developed in the research project carried out during the years 1996-1997. The project covered e.g. development of composite coatings for precurved sandwich panels under wheel loading planned to be used on a railway cargo wagon (Kujala and Marttila, 1997, Kujala, 1998b), strength analysis of laser welded crane structures (Remes and Kujala, 1997) and design and fatigue testing of longitudinal joints for sandwich panels planned to be used as deck structures on a cruising ship (Kotisalo, 1998, Kujala, 1998a, Kujala et al., 1998). In addition local strength of sandwich panels has been analysed under concentrated loads (Naar, 1997, Kujala and Naar, 1998).

In the following, the research at the Ship Laboratory of HUT related to the ultimate strength of all steel sandwich panels is shortly reviewed. The studies include laboratory strength testing, numerical FEM analysis and development of design formulations for these panels.

ULTIMATE STRENGTH UNDER HYDROSTATIC LOADING

Definition of elastic response

The linear elastic theory of sandwich structures under uniform loading is well established (Plantema 1966, Allen 1969). The deflections w_B and w_{SC} due to bending moment M(x) and shear force Q(x), respectively, are treated separately and superimposed. The total deflection is then

$$w_{TOT}(x) = w_B(x) + w_{SC}(x) \quad (1)$$

where the subscript *sc* is used to remind that it is assumed that the whole shear force is carried by the core. This model is satisfactory for engineering purposes as long as the face plates are thin and the loading is uniformly distributed, but if the face plates are thick or if the local response in the face plates is of special interest, a refined model is necessary. This can be derived by taking the bending and shearing stiffnesses of the face plates themselves into account. For example Van der Neut (Plantema 1966), Allen (1969) and Tuhkuri (1996) have analysed this problem. When the stiffnesses of the face plates are taken into account, the face plates alone carry some portion of the load. This introduces an increase in the stresses at the face plates and a decrease in the displacements.

The elastic constants, like bending and shear stiffnesses, for sandwich structures are calculated by considering the panel as a composite structure, possibly built of more than one material. The corrugated core sandwiches dealt with in this paper are made from only one material (steel) which simplifies the calculation. The elastic constants for these structures can be derived from the equations given by Libove and Hubka (1951). It can often be assumed that the shear stiffness of a corrugated core panel is large enough to be considered infinite.

Evaluation of ultimate strengh

The failure criteria of corrugated core sandwich panels are not as well established as the linear stress analysis methods. As these structures are constructed from thin plates, local instabilities of a face or core plates must be considered as possible failure modes. Under bending one of the face plates and part of the corrugated core are under compression and can buckle. In-plane compression and local loading lower this buckling load. The ultimate load carrying capacity is set by the formation of a plastic hinge. Figure 1 shows the buckling mode considered here: buckling of a face plate between two corrugations.

By modelling the face plate between two corrugations as a long plate, a first approximation of the elastic buckling stress can be calculated from the Bryan's formula (Hughes 1988)

$$\sigma_e = k\frac{\pi^2 D}{b^2 t_f} \qquad ;\frac{\sigma_e}{\sigma_y} \leq 0.5 \tag{2}$$

where D is flexural rigidity of the plate, k is a constant depending on the boundary support (k=4 for a simply supported plate), and σ_y is the yield strength of the material. For sturdy plates the failure strength can be estimated with the Johnson-Ostenfeld equation (Hughes, 1988):

$$\sigma_{cr} = \sigma_y - \frac{\sigma_y^2}{4\sigma_e} \qquad ;\frac{\sigma_e}{\sigma_y} > 0.5 \tag{3}$$

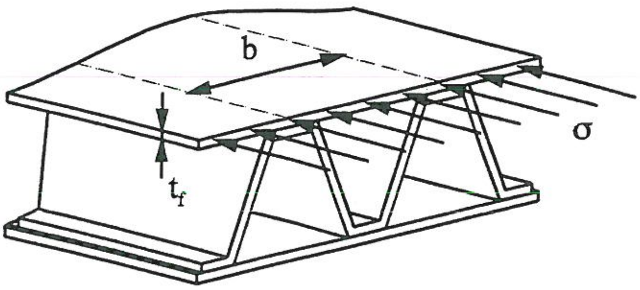

Figure 1. In plane compression of face plate.

The moment to cause formation of a plastic hinge can be calculated by standard methods (see e.g. Hughes, 1988). However, buckling of a face plate should be taken into account in the calculation. This can be estimated by using an effective breadth b_e instead of the geometric breadth b when calculating the cross sectional properties. The effective breadth can be calculated from the Faulkner formula (Hughes 1988).

Comparison with laboratory experiments

Four sandwich panels with equal dimensions have been tested under hydrostatic loading (Kujala et al., 1995). Three of the panels had the corrugation in the longitudinal direction, and one in the transverse direction. All the panels were manufactured by laser welding. A special test arrangement was developed to obtain uniform hydrostatic pressure on the panel lying between two hinged supports as shown in Figure 2a. The nominal length of the tested panels was 2500 mm, the breadth at the top face plate 340 mm and at the bottom face plate 280 mm. The height of the panels was 55 mm and thicknesses of the face and core plates were 1mm and 0.7 mm, respectively. The loading on the test panels was gradually increased with the aim of causing the collapse of the test panels. The deflection of the panels was measured with 10 displacement gauges and 20 strain gauges were installed on the panel to study the stress state during the loading.

Figure 2b shows the measured load versus deflection curve for the three tested panels with the corrugation along the long edge. The deflection given in Figure 2b was measured at midspan. The calculated limit loads of the structure was obtained following the equations described above. The yield strength of the steel used was also measured. It wa 153 MPa for the face plate material and 184 MPa for the core material.

As shown in Figure 2b, the calculated load for formation of a plastic hinge follows closely the measured ultimate strength. The collapse of the panels initiated as compressive failure of the top plate. Thereafter the corrugated core buckled following the buckling form of the top plates. Due to the properties of the weight optimised cross section, the ultimate strength is reached close to the yielding of the material. The final collapse takes place when a plastic hinge develops at the midspan of the beam. The square shape of the buckling waves on the top and corrugated core were clearly seen on the tested panels after the tests. No cracks or other damage were observed on the laser welded joints.

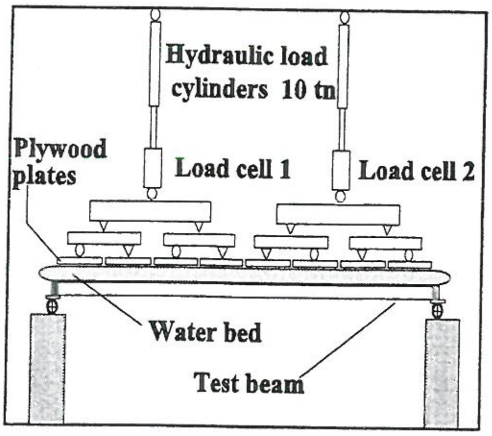

Figure 2. a) Illustration of the test arrangements for the strength tests under hydrostatic loading (Kujala et al., 1995).

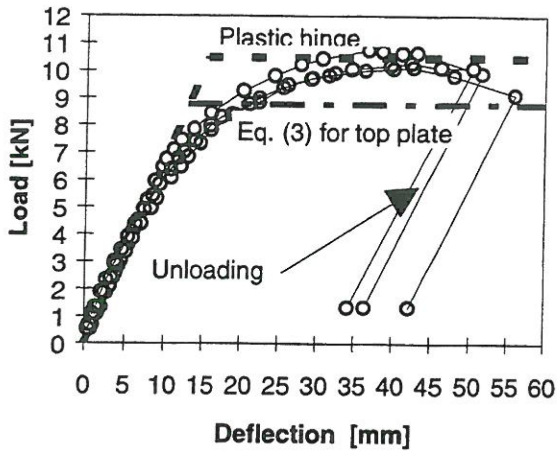

Figure 2. b) Comparison of the measured limit states with calculated limit values (Kujala et al., 1995).

ULTIMATE STRENGTH UNDER LOCAL LOADS

Available design methods

Roberts (1983) and Roberts and Newark (1997) have developed simple mechanism solutions for predicting ultimate loads for girders under patch loading. The mechanism solution for web buckling is based on the formation of plastic yield lines in the web plate and plastic hinges in the flange, see Figure 3. Thereafter the ultimate load can be calculated from an energy equation. The energy equation is obtained by equating the external work made by the load and the internal plastic energy.

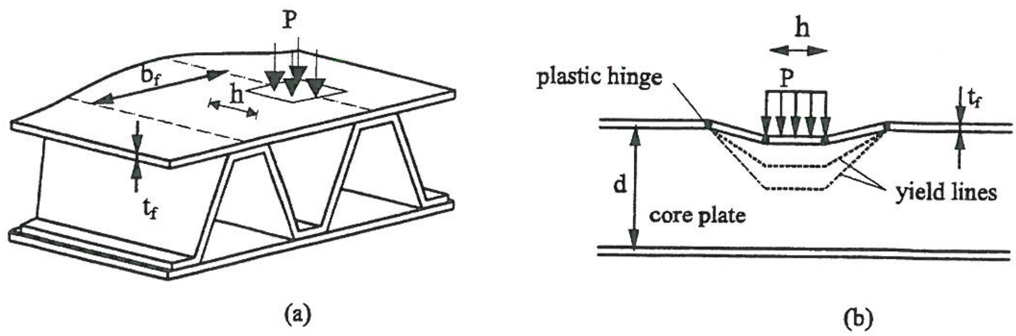

Figure 3. Assumed collapse mechanism by Roberts (1997) for I-beams.

Roberts has given two equations for the ultimate load, the first published in 1983 and the second in 1997. The basic forms of the equations are based on the analysis of the mechanism solution and thereafter some complex parameters are adjusted using observations of numerous tests on slender plate girders. The formulations are specified to give lower-bound solutions for the collapse load. The web buckling can be assumed to be similar for sandwich panels and in the following these equations are used for comparison purposes so that the load is multiplied by 2 to take into account that there are 2 web plates on the corrugated core under the local load, see Figure 3. Then the equations for the ultimate load P_u get the form :

$$P_u = t^2{}_c\left\{E\sigma_{yc}\frac{t_f}{t_c}\right\}^{0.5}\left\{1+\frac{3h}{d}\left(\frac{t_c}{t_f}\right)^{1.5}\right\}\left\{1-\left(\frac{\sigma_b}{\sigma_{yc}}\right)^2\right\}^{0.5} \tag{4}$$

$$P_u = \frac{2}{F}\left[1.1t^2{}_c(E\sigma_{yc})^{0.5}\left(\frac{t_f}{t_c}\right)^{0.25}\left(1+\frac{h\,t_c}{dt_f}\right)\right]\left\{1-\left(\frac{\sigma_b}{\sigma_{yc}}\right)^2\right\}^{0.5} \tag{5}$$

where d is height of the I-beam, h is load height, E is elastic modulus, t is plate thickness and σ_y is yield stress with subscripts c and f refering to core and face plate and σ_b is the stress due to the global bending of the panel under consideration. On equation (5) F is a safety factor, which is recommended to taken as 1.45. Equation (4) is published on 1983 and equation (5) on 1997.

B6 is a Finnish building rule for thin steel plate constructions (Thin steel structures, 1989). The rule gives an empirical equation which is intended to estimate the strength of corrugated plates under point loading.

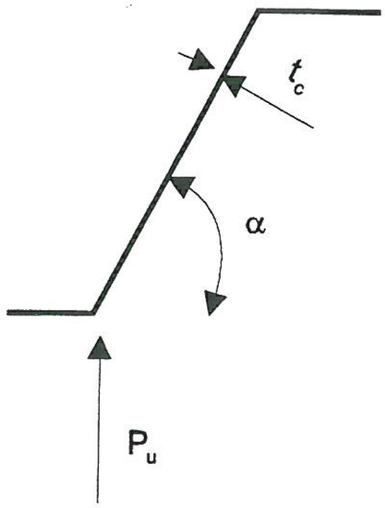

Figure 4. Definition of corrugated core according to B6 (Thin steel structures, 1989).

For the core shown in Figure 4, the resistance to point load can be calculated from the equation:

$$P_u = 2(1+0.01\frac{h}{t_c})\sigma_{yc}t^2{}_c(4.3-765\frac{\sigma_{cy}}{E})(2.4+\left(\frac{\alpha}{90}\right)^2) \qquad (6)$$

Here also the load is multiplied by 2 to take into account that in the sandwich structure shown in Figure 3, there are 2 web plates under the load.

Development of ultimate load formulations by Naar

Naar (1997) has studied the collapse mechanism of the corrugated core sandwich assuming the deformation shape illustrated in Figure 5.

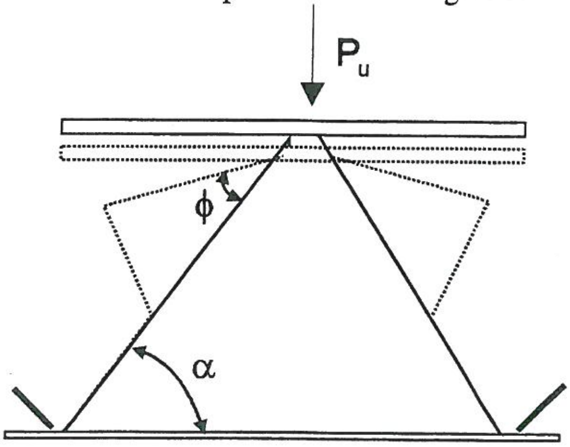

Figure 5. Assumed deformation by Naar (1997) for collapse of the sandwich structure.

Using the collapse mechanism shown in Figure 5 and applying similarly to Roberts (1997) the yield line approach, Naar developed the following formulations to estimate the ultimate patch load:

$$P_u = 2(\frac{4M_f}{\beta}+\frac{4M_c}{\beta}(\frac{k_1\beta h+\frac{M_f}{2M_c}}{1+k_1At_c})(1-(\frac{\sigma_b}{\sigma_{yc}})^2)^{0.5} \qquad (7)$$

where the variables are :

$$M_f = \sigma_{yc}b_f t^2{}_f/4 \quad ; \quad M_c = \sigma_{yc}t^2{}_c/4 \quad ; \quad I_f = b_f t^3{}_f/12$$

$$k_1 = \frac{\sigma_{yf}\sqrt{\sin\alpha^2-\sin\phi^2}}{40t_c\sigma_{yc}\sin\phi\cos\phi} \quad ; \quad \phi = \arctan(2k_2\frac{\sin\alpha^2}{\sin\alpha^2-k^2{}_2});$$

$$k_2 = \frac{M^2{}_f}{3EI_f 4M_c}$$

$$\beta = \sqrt{\frac{M_f}{4M_c k_1}}$$

The variables used in equation (7) are specified in Figures 3 and 5.

Description of the laboratory experiments used to verify the design methods

Naar (1997) conducted laboratory experiments with the equipment shown in Figure 6. A local load was simulated by a metallic tube which had the function to divide the force on the defined area. By changing the tube diameter, the load height could be varied. The sandwich panels were placed on rolls to create simply supported end conditions. The load and displacements were measured as shown in Figure 6. The displacements were measured relative to the upper face plate of the panel just above the supports. The measured deflection includes the local deflection and also the global deflection due to bending.

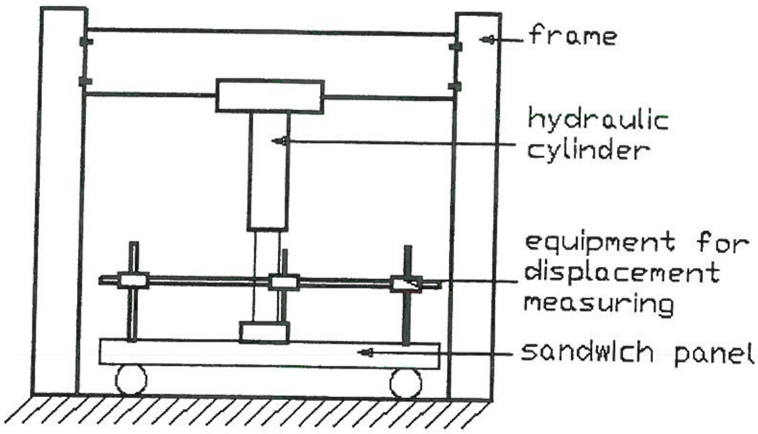

Figure 6. Layout of the equipment for sandwich panels local strength testing (Naar, 1997).

All the sandwich panels tested by Naar (1997) were 1000 mm long steel sandwich structures with corrugated cores. Face plate and corrugated core thicknesses were varied from 1 to 3 mm. The angle between the corrugated core and face plate was 60 degrees and the height of panel was 100 mm. In addition, two panels with 2mm face and core thicknesses, 40° and 60° corrugation angles and 100 mm and 150 mm heights, were tested. The tests were made with three different patch load areas by using tube diameters of 20, 60 and 100 mm. Loads were located at midspan or at the distance of 300 mm from the panel edge in longitudinal direction. Table 1 summaries the dimensions of the tested panels. In Table 2, the material properties for the used plates are represented. The yield strength varied from 256 MPa to 407 MPa.

Table 1. Summary of the tested panels. Figures 3 and 4 show the geometry of the panels.

Height d [mm]	Face plate tf [mm]	Core plate tc [mm]	Lower plate tl [mm]	Corr. angle α [o]	Panel number
100	1	1	1	60	Naar 10
100	2	2	2	60	Naar 7
100	3	3	3	60	Naar 11
150	2	2	2	60	Naar 9
100	2	2	2	40	Naar 12
100	1	1.5	2	60	Naar 2
100	2	1.5	2	60	Naar 1
100	2.3	1.5	2	60	Naar 4
100	3	1.5	2	60	Naar 5

Table 2. Material properties for the tested plates.

Steel	Plate thickness [mm]	Yield strength [MPa]	Tensile strength [MPa]	Elongation %
Racold 320 HSF	1	407	485	36
Racold 320 HSF	1.5	370	448	38
Ragal 280 100 MA	2	307	398	34
FORM 300 C	2.3	256	337	36
Racold 280 HSF	3	285	391	34

Description of the FE-calculations used to verify the design methods

The ultimate local strength for the panels tested by Naar was studied also with a FE model using the same load configurations and structural dimensions as used in the laboratory tests. A nonlinear FE model was created by using two-dimensional shell elements. The geometry was created with I-DEAS software and using I-DEAS converter the geometry was transformed into ABAQUS input file format. The final results were obtained with ABAQUS solver and analyzed using ABAQUS post programs.

In the modelling, six node triangular shell (STRI65) and eight node quadrilateral shell (S8R) elements were used. As the stress disdribution is non-linear in the cross-section, seven integration points through the thickness were used.

The model shown in Figure 7 was made to study the behaviour of the laboratory test specimen of Naar (1997). As the local strength under global bending is related with bending stresses, the model must include properly also the global bending behaviour. Therefore the model had the same breadth as the laboratory test specimen. Because the sandwich panel has two symmetric planes, only 1/4 of the sandwich panel was modelled.

The material was considered nonlinear except the elements on the lower plate near the support and on the left border, far from the load area, where the material was assumed to behave linearly. The corrugation angle was 60 degrees. The panel height was 100 mm. The patch load was modelled as a pressure load on the elements.

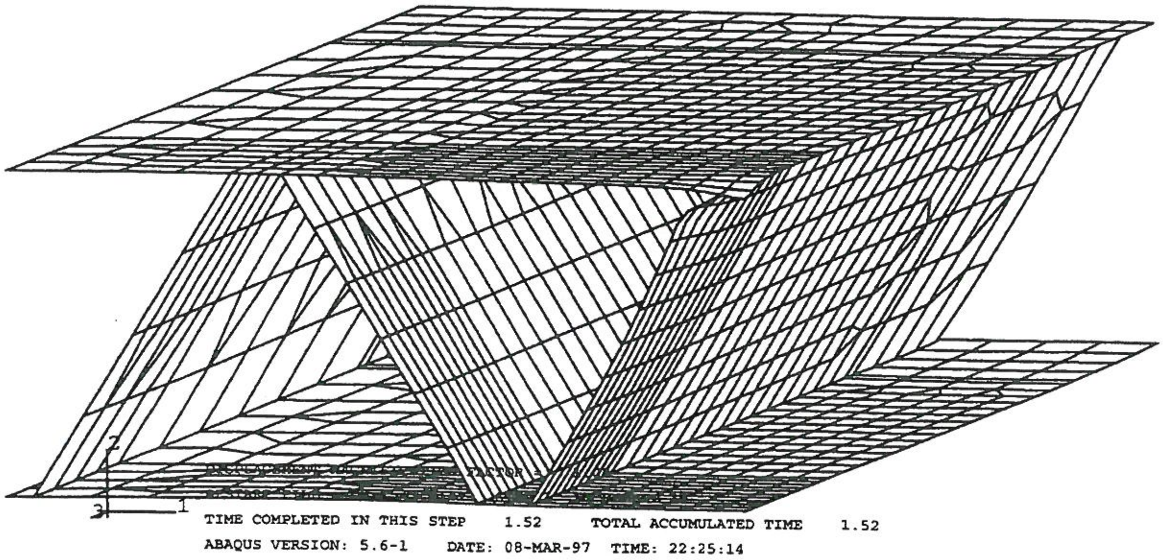

Figure 7. FE-Model with 1677 elements, 5045 nodes and with 30270 dof (Naar, 1997).

Comparison of laboratory tests and FE-calculations results with design formulations

Figure 8 gives examples of the measured and calculated ultimate strength values for the sandwich panels under patch loading with load heigth of 60 mm. The face, core and lower plate thicknesses varied from 1 to 3 mm. As can be seen from Figure 8, Naar design approach follows well the measured data whereas the Roberts equations give 50-70 % higher values and standard B6 gives 40 % lower values than those measured.

A more detailed comparison of the results obtained has been made by Kujala and Naar (1998) . The results indicate, however, that the accuracy of the studied design equations increase with increasing plate thickness and load height. Naar (1997) formulations give the most reliable estimate for the ultimate strength, the accuracy being typically better than 15 %. The only exception is when load height is 20 mm and all plate thicknesses are 1 mm, as then the Naar formulations give about 50 % too high values for the local strength. In this lower load height and plate thickness category the building rules B6 seems to be the most reliable design load estimation method. The building rules can not be applied, however, if the face plate thickness is so thick that it has also effect on the ultimate strength.

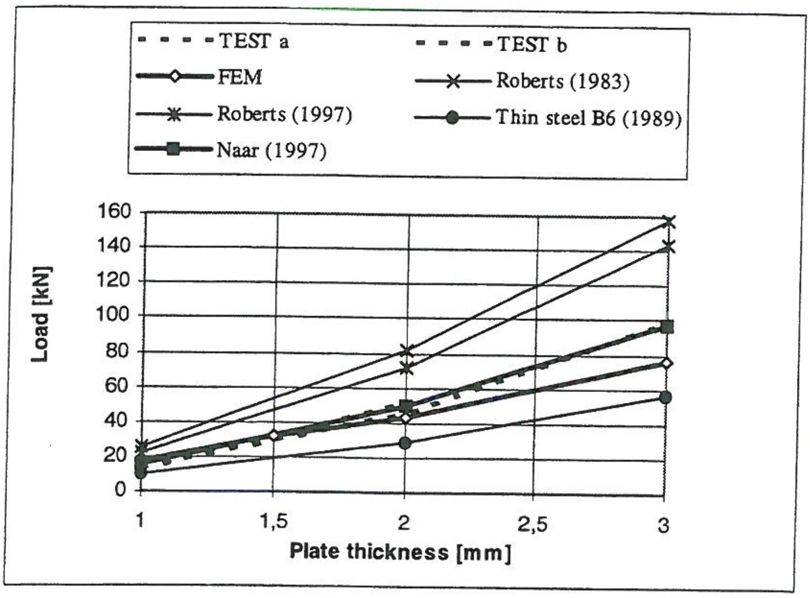

Figure 8. Measured and calculated ultimate load values for the studied sandwich panels as a function of used plate thicknesses. Load height is 60 mm. Face, core and lower plates of the sandwich panels have the same thickness.

CONCLUSIONS

In this paper, the work carried at the Ship Laboratory of Helsinki University of Technology to study the ultimate strength of all steel sandwich panels has been reviewed. Under considerations have been panels with height varying from 50 to 150 mm and with plate thickness varying from 1 to 3 mm. Several experiments and numerical stability analyses have been conducted and compared with existing design approaches for plate buckling in compression, with recommendations B6 for thin steel structures, with Robert's formulations for localized edge loading of slender plate girders, and with Naar formulations specifically developed for corrugated core sandwich panels under local loads.

Three cases can be classified for the collapse modes. For large loading areas and for small core plate thicknesses, elastic bucklingof the core plate is the dominating collapse mode. For thicker plates, core yielding and buckling are causing the failure. With shorter load heights the panel geometry has smaller effect and the core plate thickness and the yield strength dominates the collapse behaviour. The third type of collapse mode occurs when the face plate is thin. Then the applied ultimate load causes high compressive bending stresses on the face plate and upper part of the core plate. This can cause face plate buckling before the collapse of the core plate.

The buckling of face plates under compression can be analysed with standard methods available. The strength under local loads is more problematic. Naar formulations give the most reliable estimate for the ultimate strength level under local loading, the accuracy being typically within the region of 10-15 % with plate thickness thicker than 1.0 mm and load height higher than 60 mm. Finnish recommendations for thin steel structures B6 gives typically 40-60 % smaller values than the measured figures except for the lowest plate thicknesses and load height, when the accuracy of B6 is better than 5 %. This is explained by the fact that B6 is developed for point load applications on thin steel corrugated cores. The standard B6 excludes the effect of face plate so it can not be applied with thicker face plates. The Roberts formulation developed for slender girders give about 50-60 % too high ultimate strength estimates for sandwich panels with corrugated cores.

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ENGLISH SUMMARY

FATIGUE STRENGTH OF LONGITUDINAL JOINTS OF CORRUGATED CORE STEEL SANDWICH PANELS

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The article discusses the fatigue strength of longitudinal joints of ships all metal sandwich panel decks. Comparison with different joint types is made by using finite element analysis. For two different joint types SN-curves are determined experimentally. These SN-curves are used for calculating the damage ratio for a cruise ship superstructure. Calculations revealed that beside of the few highest decks the damage ratio will remain below one.

ILMOITUSASIAT
Announcements

Tieteellisten Seurain Valtuuskunnan TSV kokous, Vuoden Tiedekirja 1997-palkinto ja -kunniamaininnat sekä Tutkimuseettinen seminaari 27.3.1998.

Tieteellisten Seurain Valtuuskunta piti vuosikokouksensa 27.3.1998 Tieteiden talolla (Kirkkokatu 6, 00170 Helsinki). Säätytalon siirryttyä ensisijaisesti valtioneuvoston edustustilaksi osoitettiin tieteellisten seurain käyttöön peruskorjattu Kirkkokatu 6 eli Tieteiden talo, jossa joillakin seuroilla on toimistonsa ja jäsenseurat voivat pitää siellä kokouksiaan.

TSV:n hallituksen kokoonpanoon tuli pieniä henkilömuutoksia, mutta hallituksessa edustettuina olleet seurat säilyivät ennallaan. Toimintakertomus ja tilit vuodelta 1997 hyväksyttiin sekä myönnettiin vastuuvapaus hallituksen jäsenille ja toiminnanjohtajalle. Talousarvio ja toimintasuunnitelma vuodelle 1999 hyväksyttiin. Uudeksi jäsenseuraksi hyväksyttiin hallituksen esityksestä **International Fiscal Association, Suomen Osasto ry**.

Ensi vuonna tulee kuluneeksi 100 vuotta Tieteellisten seurain valtuuskunnan perustamisesta. Ensi vuosi on *tieteellisten seurojen juhlavuosi*, jonka yleisotsikkona on *Tiede luo tulevaisuuden*. Juhlavuonna toteutetaan *Suomen tieteen historia*-projekti ja ilmestyy *tieteellisten seurojen hakuteos*, johon on kerätty tietoja jäsenseuroista. *Tieteen päivät* 99 toteutetaan 13.-16.1.1999 otsikolla *Matkalla tulevaisuuteen.*Vuoden *pääjuhla* on 2.10.1999. Lisäksi järjestetään erillisiä näyttelyitä ja seminaareja.

Valtuuskunnan kokouksen jälkeen iltapäivällä julkistettiin **Vuoden Tiedekirja 1997 -palkinto ja -kunniamaininnat:**

> Vuoden Tiedekirja 1997-palkinnon sai toimittaja **Juhani Mänttäri** ja Suomen Biologian Seura Vanamo teoksesta ***Suomen luonnon sata vuotta***

Kunniamaininnan saivat

Harri Kalha: *Muotopuolen merenneidon pauloissa. Suomen taideteollisuuden kultakausi: mielikuvat, markkinointi, diskurssit.*

Jussi Nuorteva: *Suomalaisten opinkäynti ennen Turun Akatemian perustamista 1640.*

Arne Rousi: *Auringonkukasta viiniköynnökseen.* Ravintokasvit.

Päivän viimeisenä tapahtumana oli **Tutkimuseettinen seminaari**. Kolmen ansiokkaan alustuksen jälkeen keskusteltiin. Käsiteltyjä aiheita olivat tutkijan mahdollisesti harjoittama väärennös, plagiointi, vilppi ja huono käyttäytyminen sekä miten näihin tulee puuttua. Lisäksi keskusteltiin tutkimuksen teettäjän ja rahoittajan roolista. Kevään kuluessa Tutkimuseettinen neuvottelukunta julkaisee näistä aiheista kolmiosaisen ohjeiston. Siihen on kaikkien syytä tutustua.

Matti A Ranta