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Autor: Takahashi, Yasuhiko / Takemoto, Yasushi / Takeda, Toshikazu

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Experimental Study on Thin Steel Shear Walls and Particular Steel Bracings under Alternative Horizontal Load

Etude expérimentale de parois de cisaillement minces en acier et d'entretoisements particuliers soumis à des forces horizontales alternées

Experimentelle Studie über dünne Stahlblech-Schubwände und spezielle Stahlaussteifungen unter alternierender horizontaler Last

Yasuhiko TAKAHASHI Toshikazu TAKEDA
Yasushi TAKEMOTO Masatoshi TAKAGI
Researchers of Structural Engineering Laboratory
Technical Research Institute
Ohbayashi-Gumi Ltd.
Tokyo, Japan

In seismic countries as well as in Japan, structures are designed against earthquakes and reinforced concrete shear walls or steel bracings are usually used as aseismic resistant element. However, their hysteretic characteristics in plastic region, ductility and capacity of energy absorption are not always good. Besides, their stiffness is so rigid that structure designed by static analysis is occasionally disadvantageous, when dynamically analized.
The authors devised the thin steel plate shear wall and the particular steel bracing system with stable behavior and investigated their behavior through the tests whether they can be actually practised.

1. Thin Steel Plate Shear Wall

The steel plates are so strong and ductile, and their weight is so light that they are suitable as a material of shear wall. However, they have defect of buckling under low stress level. To improve this characteristic, the method of welding rib plates on one or both sides of steel plate was adopted.

First, the basic test was done, the object of which was to obtain the design principles of rib plates to be provided with consideration of calculated results. Changing the spacing between rib plates and their stiffness, the strength, hysteresis loop and post buckling behavior of steel shear wall were investigated.

Next, based on the design principles obtained through the basic test, the steel plate shear walls of the 32-storied building were designed for actual use and two full-size models of them were examined through the tests.

1.1 Basic Test

Outline of test---Twelve specimens with 2.3, 3.2 or 4.5 mm in plate thickness were made. They were reinforced with various sectional rib plates whose forms of arrangement were G, M1, and M2 type shown in Fig.1. The name of specimens, for example PR-3.2-M2-15, means the abbreviation of steel plate with rib plates, plate thickness, form of arrangement and height of rib plate in order (Table 1).

A very stiff rectangular rigid frame connected with pin joints, was attached with high tensile bolts arround the specimen. To apply the shear force to the specimen, compressive force was subjected in one diagonal direction of the frame (Fig.2). When deflection was reached some decided values, the specimen was turned arround to be subjected to the force in another diagonal direction.

Calculation---The rigidity and yielding stress (τ_{sy}) on shear theory were calculated by eq.1 and 2. The rigidity, yielding stress (τ_{ty}) and maximum stress (τ_{tmax}) on tension field theory were calculated by eq.3,4 and 5.

$$\tau = G\cdot\gamma \quad \cdots (1) \quad , \quad \tau_{sy} = \sigma_y/\sqrt{3} \quad \cdots (2)$$

$$\tau = E\cdot\cos^2\alpha\cdot\sin^2\alpha \quad \cdots (3) \quad , \quad \tau_{ty} = \sigma_y\cdot\cos\alpha\cdot\sin\alpha \quad \cdots (4) \quad ; \quad \tau_{tmax} = \sigma_{max}\cdot\cos\alpha\cdot\sin\alpha \quad \cdots (5)$$

Where E,G, γ and α denote Young's modulus, shear modulus, shear strain and angle of buckling wrinkle respectively. σ_y and σ_{max} are yielding stress and maximum stress of the material.

The stress (τ_{cr}) at partial buckling (buckling of steel plate enclosed with vertical and horizontal rib plates) was calculated by eq.6 and the stress at entire buckling of the wall by eq.7. It was assumed that steel plate was connected to the surrounding frame with pin joints.

$$\tau_{cr} = \pi^2\cdot k\cdot E\cdot t^2/12\cdot(1-\nu^2)\cdot b^2 \quad \cdots (6)$$

$$\left.\begin{aligned} & U + V_p + V_r = const. \\ & U = -\tau\cdot t\int_0^a\int_0^b \frac{\partial w}{\partial x}\cdot\frac{\partial w}{\partial y}\,dx\,dy \qquad V_p = \frac{D}{2}\int_0^a\int_0^b\left(\frac{\partial^2 w}{\partial x^2}+\frac{\partial^2 w}{\partial y^2}\right)^2 dx\,dy \\ & V_r = \sum_i \frac{B_{xi}}{2}\int_0^a\left(\frac{\partial^2 w}{\partial x^2}\right)^2_{y=y_i} dx + \sum_j \frac{B_{Yj}}{2}\int_0^b\left(\frac{\partial^2 w}{\partial y^2}\right)^2_{x=x_j} dy \\ & w = \sum_m\sum_n a_{mn}\cdot\sin\frac{m\pi x}{a}\cdot\sin\frac{n\pi y}{b} \end{aligned}\right\} \cdots (7)$$

where t, a and b denote plate thickness, width and length, and U, Vp, Vr and potential energy of force, strain energy in plate and in rib plates.

Test result and discussion---Fig.3 showed that the initial rigidity agreed well with eq.1. Then in the steel plates of P-2.3, PR-3.2-M2-15, PR-4.5-M1-15, and PR-4.5-G-10, the rib plates of them were so small that entire buckling occurred in elastic region. In the specimen PR-2.3-M2-60, partial buckling occurred because the spacing of rib plates was so large and buckling wrinkles began to appear at the stress nearly equal to the calculated. After that the rigidity gradually decreased as the plates transferred to tension field. Accordingly their yielding stress was obtained by eq.4 and the hysteresis loop was S-shape for the load reversals at small amplitude of deflection.

The rigidity of PR-4.5-M1-35 and PR-4.5-M1-55, buckled at the stress of about proportional limit, and of the other specimens in which plastic buckling occurred, began to decrease at 0.7τ_{sy}. Their yielding stress could be obtained by eq.2. The hysteretic characteristics of the former was spindle shape when the deflection was small, but S-shape once the deflection got large. One of the latter was spindle shape which could be modified simple bi-linear relation.

In final stage, large buckling wrincles appeared in all specimens (Fig.4) and maximum stresses obtained with tests was from 0.74 to 0.9 τ_{tmax}. The theoretical maximum stress would be regarded as τ_{tmax} at which tensile break occurred. All specimens had very large ductility and some of them deformed to the extent of γ = 0.1 radian.

The test results showed that the steel plate with rib plates on the both sides had more stable behavior than one reinforced on one side only, and there was no difference of hysteresis loop because of the forms of arrangement of rib plates.

Design principles for rib plates---In order to get the steel shear wall with stable spindle-shaped hysteretic characteristics and with deformability of 0.01 radian to shear stress, next two design principles were recommended through the test results.

i) Shear wall shall not buckle in elastic region.

ii) At final stage partial buckling rather than entire buckling shall occur.

For item i), spacing and stiffness of rib plates shall meet eq.6 and eq.7. For item ii), stiffness of rib plates shall have somewhat two times larger stiffness than the value requested by eq.7.

1.2 Full Size Test

Outline of test---Fig.5 shows two specimens of one bay-two storied steel plate wall, F-1, the specimen without opennings in the wall and F-2, the specimen with two opennings. F-1 was provided with 4.5 mm steel plate in thickness. In order to give same shear rigidity and strength as that of F-1, F-2 was provided with 6 mm steel plate in thickness. These specimens, the basis of which were fixed, were tested under the alternative horizontal load. Fig.6 shows the view at test.

Analysis---The theoretical analysis was done by elasto-plastic finite element method (F.E.M) under the following assumption: The steel plate wall never buckles, and material has bi-linear stress-strain relationship under Von Mises' criterion of yielding. Otherwise the rigidity and the yielding load were calculated by eq.1 and 2.

Test results---F-1--Horizontal load(P)-deflection(δ) curve, shown in Fig.7, was linear before P reached 250 tons and agreed with the results by eq.1 and F.E.M. Then the rigidity gradually decreased because of local yielding and subsequent yielding by shear force. As the load increased, the yielding zone spreaded and meanwhile partial buckling phenomena were observed at several points. There was indication of entire buckling in the two-storied wall at 350 tons, due to the poor lateral constraint to the beam at second floor. If the constraint of the floor slab was considered, it wouldn't supposedly occur and the calculated results by F.E.M. would agree better with the test results.

F-2--The initial rigidity could be calculated by eq.1 or F.E.M. as in F-1. The behavior was very much influenced by the exsistence of opennings and the first yielding appeared in the web of beam between opennings. The next yielding appeared both in the corner of wall and in frame member arround them. Before the yielding spreaded wide into steel plate, the plate buckled partially arround the opennings. The influence of this buckling on the rigidity, however, was so small that the results by F.E.M. was in good agreement with the test results even in plastic region.

Judging from the test results of both F-1 and F-2, it would be satisfactory to regard the shear stress obtained by eq.2 as the yielding stress of the wall and to consider that both specimens had stable hysteresis loop and large ductility. In these test, break of the welding at the bottom of the column made the continuation of loading stop.

1.3 Conclusion

The steel plate shear wall with rib plates designed on the basis of the principles obtained by the basic test, had very good behavior and both its rigidity and strength could be calculated by shear theory. The rigidity and strength of the steel plate with an openning was supplied well with increase of plate thickness and adequate reinforcement arround it.

2. Particular Steel Bracing System

In braced frames, once the bracings buckle, the load comes down. Accordingly the ductility and capacity of energy absorption decrease to the successive load reversals. Therefore the authors tried to keep bracing system stable to the horizontal load on the basis of the two principles: Never make the bracing buckle and make only the part of the system yield. This report describes the particular bracing systems, namely tension-yield type bracing, beam yielding type, or partial yielding type of bracing in which yielding was intended to occur much earlier than the attainment of buckling of bracing.

Specimens---Fig.8 shows four specimens. SK type has standard bracings. As bracings of STK type are reinforced with 10 cm x 10 cm sectional reinforced concrete, the tensioned bracing yields first. Large eccentricity is purposely given to the bracings of BBK and SPK, and the beam or additional member that connects bracings to beam, is intented to yield before the bracings buckle.

Calculation---The specimens were analized to horizontal load in elasto-plastic region, considering such property as in Fig.9 to each member.

Test Results---Load-deflection curve in Fig.10 showed that the post buckling behavior of SK type was not good, as written in Introductory Report. The decrease of load appeared in the calculated results too.

As the reinforced concrete prevented the bracing from buckling and increased the compressive strength, the rigidity and strength of STK could be larger than ones of SK and the hysteretic characteristics was more stable. However, the concrete collapsed finally and load came down. The calculated results traced well the sequence of yield occurrence of members.

In SPK type, the first yielding was observed in the additional member at 15.2 tons and the rigidity decreased abruptly. The hysteresis loop was as stable as the steel plate shear wall. The calculated results agreed well with the tested before γ reached 1/125, where shear buckling appeared in the web of additional member. Though, by analysis, yield hinges were formed at the top and the bottom of columns, they had little influence on the deformation of the frame.

The strength of BBK was determined by the yielding of beam. The test proved that the first yielding of beam caused the decrease of the rigidity. Concerning of hysteretic characteristics and ductility, BBK was as good as SPK, however attention might have to be paid in the vertical deflection of beam. Fig.11 shows the specimens after testing.

The results showed that the behavior of these three new bracings was quite different from the standard one. Concretely speaking, the rigidity was freely changable in elastic and plastic region, the hystersis loop was spindle one and the deflection capacity was large. The behavior could be predicted well by the calculation under the assumption that each member had bi-linear stress-strain relation.

3. Conclusion

Finally both the thin steel plate shear wall and the particular steel bracing systems had large capacity of energy absorption and stable hysteresis loop which could be defined by the simple model. With use of these systems in the structure, the inelastic dynamic response of it can be easily obtained and the safe structure against earthquakes can be designed.

Table 1 Results of Test and Calculation

NAME OF SPECIMEN	STEEL PLATE THICKNESS	RIB PLATE			MATERIAL PROPERTY		CALCULATED RESULTS				MEASURED MAXIMUM STRESS
		SECTION	FORM OF ARRANGEMENT	STIFFENESS	YIELDING STRESS	MAXIMUM STRESS	BUCKLING STRESS	TENSILE YIELDING STRESS	SHEAR YIELDING STRESS	BREAK STRESS	
	cm	cm x cm		cm^4	ton/cm^2	ton/cm^2	ton/cm^2	ton/cm^2	ton/cm^2	ton/cm^2	ton/cm^2
P -2.3	0.23	——	——	——	3.10	5.04	0.08	1.55	1.78	2.52	1.86
PR-2.3-M2-60	0.23	0.45 x 6.0	BOTH SIDES, M2	8.10	3.10	5.04	1.05	1.55	1.78	2.52	2.14
PR-3.2-M2-15	0.32	0.32 x 1.5	ONE SIDE, M2	0.36	2.80	4.51	0.50	1.40	1.62	2.26	1.72
PR-3.2-M2-25	0.32	0.32 x 2.5	ONE SIDE, M2	1.67	2.80	4.51	>1.3	1.40	1.62	2.26	1.75
PR-3.2-M2-40	0.32	0.32 x 4.0	BOTH SIDES, M2	1.71	2.80	4.51	>1.3	1.40	1.62	2.26	1.83
PR-3.2-M2-60	0.32	0.45 x 6.0	BOTH SIDES, M2	8.10	2.32	3.80	>1.3	1.16	1.34	1.90	1.71
PR-4.5-M1-15	0.45	0.45 x 1.5	ONE SIDE, M1	0.51	2.37	3.54	0.41	1.19	1.37	1.77	1.42
PR-4.5-M1-35	0.45	0.45 x 3.5	ONE SIDE, M1	6.43	2.37	3.54	1.05	1.19	1.37	1.77	1.42
PR 4.5-M1-55	0.45	0.45 x 5.5	BOTH SIDES, M1	6.24	2.37	3.54	1.04	1.19	1.37	1.77	1.48
PR-4.5-G-10	0.45	0.45 x 1.0	ONE SIDE, G	0.15	2.37	3.54	0.37	1.19	1.37	1.77	1.39
PR-4.5-G-30	0.45	0.45 x 3.0	ONE SIDE, G	4.05	2.37	3.54	>1.3	1.19	1.37	1.77	1.47
PR-4.5-G-50	0.45	0.45 x 5.0	BOTH SIDES, G	4.67	2.37	3.54	>1.3	1.19	1.37	1.77	1.51

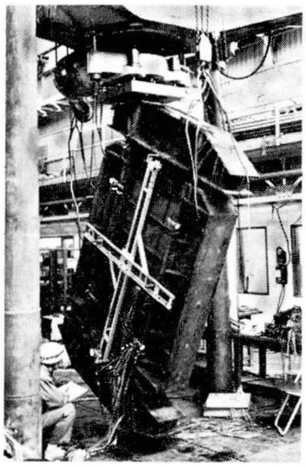


Fig.2 View at Test

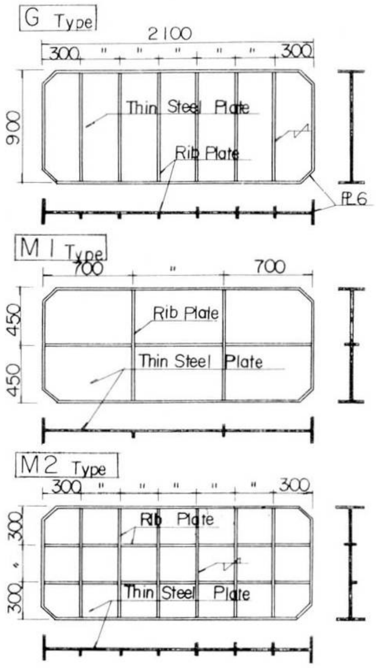
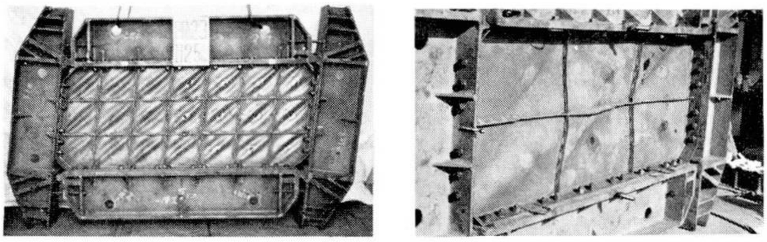


Fig.1 Test Specimens



PR-2.3-M2-60 PR-4.5-M1-35

Fig.4 Failure of Specimens

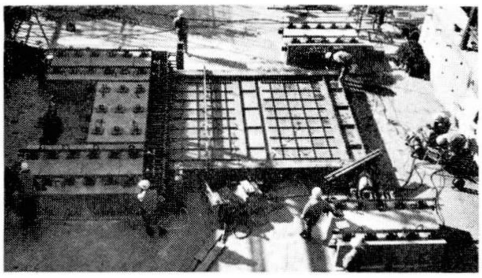


Fig.6 View at Test

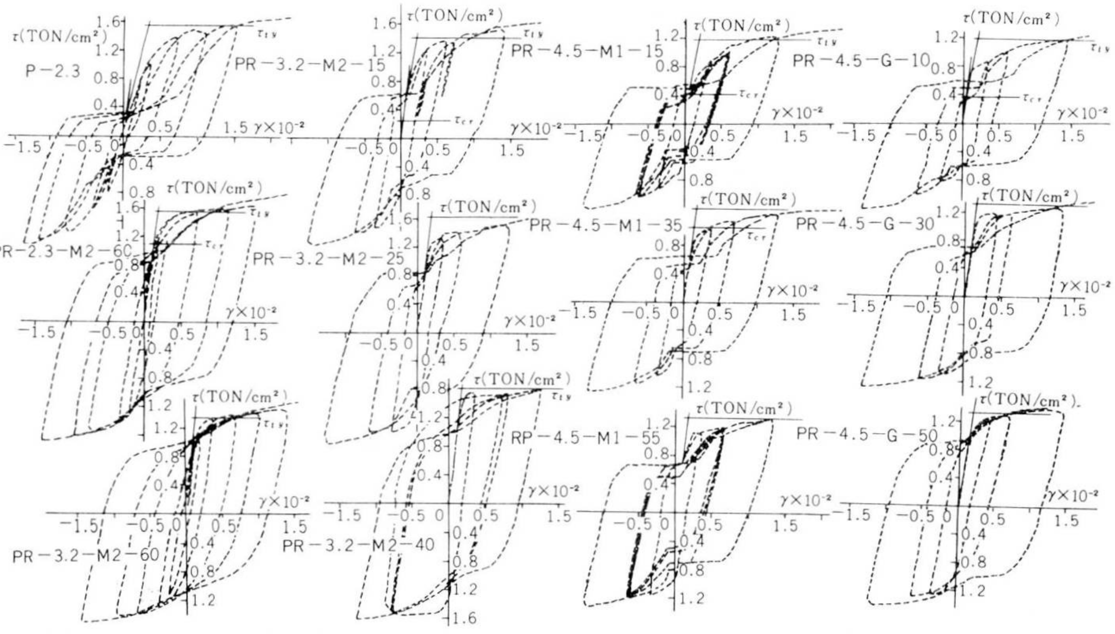


Fig.3 Shear Stress-Shear Strain Curves

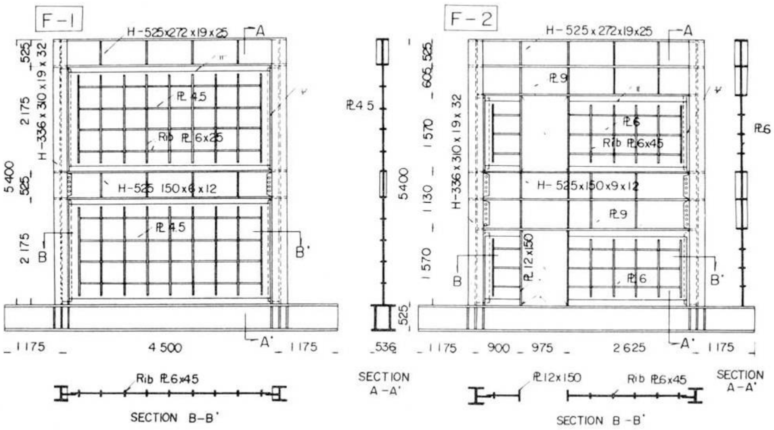
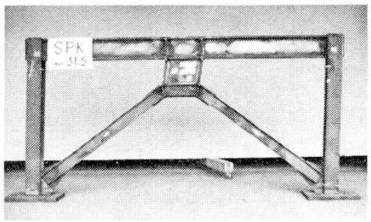
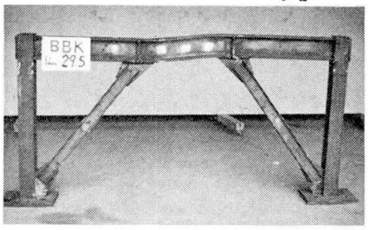


Fig.5 Specimens



SPK type



BBK type

Fig.11 Failure of Specimens

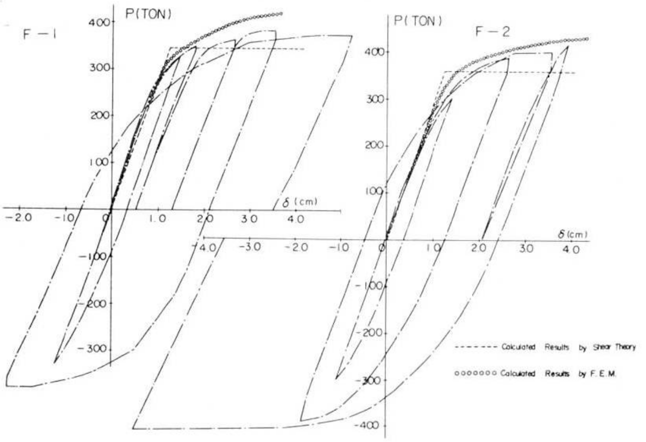


Fig.7 Load-Deflection Curves

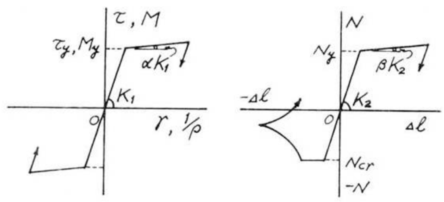
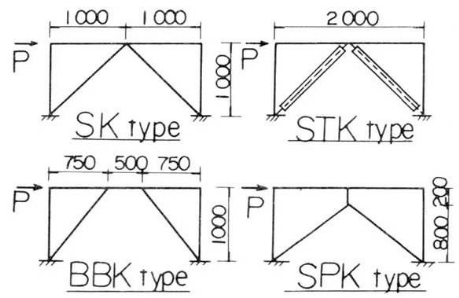


Fig.9 Property of Member



	SPECIMEN	MEMBER
COLUMN	ALL SPECIMENS	H-100x100x6 x 8
BEAM	"	H-150 x 75 x4.5 x 13
BRACING	SK , STK	2[-65 x 26 x 4.5 x 4.5
	BBK . SPK	2[-75 x 32 x 4.5 x 4.5
STRUT	SPK	H-250 x 150 x4.5 x 13

Fig.8 Specimens

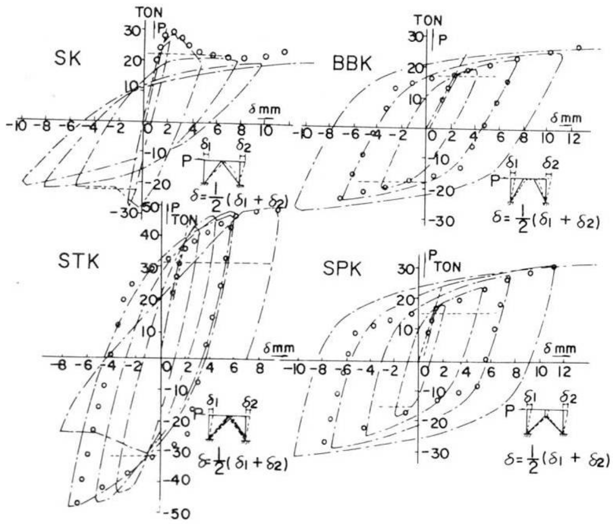


Fig.10 Load-Deflection Curves

SUMMARY

This report describes the general characteristics of the thin steel plate shear wall and the particular steel bracing systems in elasto-plastic region and the design principles to make their behavior stable, obtained through the test under alternative horizontal load and the calculated results.

RESUME

On décrit les caractéristiques générales des entretoises métalliques formées d'une plaque mince et les autres systèmes d'entretoises métalliques dans le domaine élasto-plastique, ainsi que les principes de dimensionnement permettant d'obtenir un comportement stable. On se base sur des essais de charge horizontale alternée et sur des résultats calculés.

ZUSAMMENFASSUNG

Der Bericht beschreibt die allgemeinen Charakteristiken einer dünnen Stahlscheibe und der speziellen aussteifenden Systeme im elasto-plastischen Bereich sowie die Bemessungsgrundsätze für ihr stabiles Verhalten, die sich durch Versuche unter wechselnder horizontalen Belastung ergaben, nebst den berechneten Resultaten.

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