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Development of the Fireless Reinforcement Method of H Shaped Steel Brace Joints

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Abstract

We have developed aseismic reinforcement technology of steel manufacturing equipment. I'm working on development of reinforcement technology of a steelmaking factory in recent years. A point in aseismic reinforcement of a steelmaking factory is often a brace joint. There are a lot of wiring and laying of the pipes near the joint. There is a problem with safety for use of fire of a welding. Therefore fireless and compact reinforcement method were desired. Fireless and compact reinforcement method are proposed targeted for H shaped steel brace joints by this development.

1. Introduction

In addition to the blast furnace and steelmaking plant shown in **Photo 1**, the premises of a steelworks are home to many facilities necessary for steelmaking such as a power generation plant, chimneys, and quays. To improve the aseismic capacity of these facilities, Nippon Steel & Sumitomo Metal Corporation, while introducing existing technologies, has developed original technologies. Examples of facilities in which technologies that we have developed to date for blast furnaces, steelmaking plants, and power generation plant buildings are used are shown in **Fig. 1**. For example, linked damping structures use a technology that absorbs seismic energy using hydraulic dampers by the action of horizontal displacement differences between a blast furnace itself and blast furnace scaffold during an earthquake. Buttresses and knee brace dampers are technologies that enhance the strength and absorb seismic energy while the operation space in the buildings is secured. It is important for the development of aseismic technologies to achieve structural rationality in consideration of the characteristics and restrictions possessed

Photo 1 Example of iron manufacturing equipment

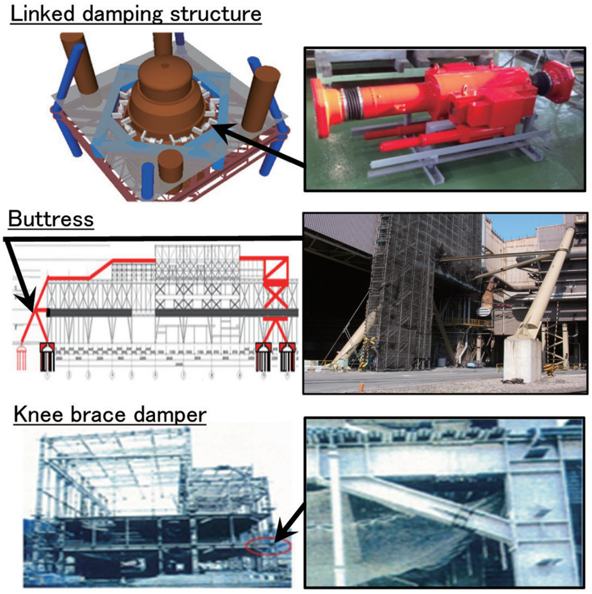

Fig. 1 Example of aseismic reinforcement technical development

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by individual steelmaking facilities. In light of this, we have continued the development of original technologies of the earthquake resistance and vibration control.

Recently, from the experience of the damage suffered during the Great East Japan Earthquake and given the increased likelihood of Nankai megathrust earthquakes, Nippon Steel & Sumitomo Metal has intensified its initiatives for aseismic reinforcement of buildings that were constructed under the earthquake-resistance standards before the revision (hereinafter referred to as “buildings under the previous standards”). Buildings under the previous standards, which have been built as production facilities, are characterized by many gas pipes and electric wires that narrow the space for reinforcement work. Therefore, reinforcement methods that can overcome such characteristics are required. In this paper, we list up issues with the aseismic retrofit of buildings under the previous standards first and then propose a reinforcement method to solve such issues. Next, we explain the confirmation results of the mechanical performance of the proposed method obtained through tests for performance checking and FEM analysis conducted towards the embodiment of the method. We also explain the confirmation results of feasibility of the method obtained through production tests, using the application of the results to actual plant buildings as examples.

2. Issues with the Improvement of Earthquake Resistance

Among buildings under the previous standards, those requiring reinforcement are top-heavy steel-frame buildings in which upper floors support heavy equipment, such as the steelmaking plant shown in **Fig. 2**. Most of the weak points of such steelmaking plants are the joints at the end of braces. There is a high possibility that during a large earthquake, these joints will become damaged before the base steel frames of the braces yield in tension and resist the earthquake. For this reason, reinforcement of these joints is the key to improving the resistance to earthquakes. In steelmaking plants in particular, H-shaped steel (hereinafter referred to as “H-beams”) is frequently used as braces, and rivets are used as joint fasteners. The method in this paper targets these types of joints with H-beams and rivets. **Table 1** shows an example of the braces targeted.

An actual H-beam joint built in a steelmaking plant is shown in Fig. 2. Since steelmaking plants are production facilities, they have many gas pipes and electric wires in the vicinity of joints, which raises safety concerns when welding and performing other operations that involve an ignition source near the joints. Given this, a reinforcement method without using an ignition source is strongly required. In addition, as in many cases operations are afforded narrow spaces around the joints, retrofitting parts need to be compact. **Table 2** shows conventional technologies that can be used to reinforce a joint. However, welding of an existing splice involves use of an ignition source, so this is not suitable. Welding of a new steel plate

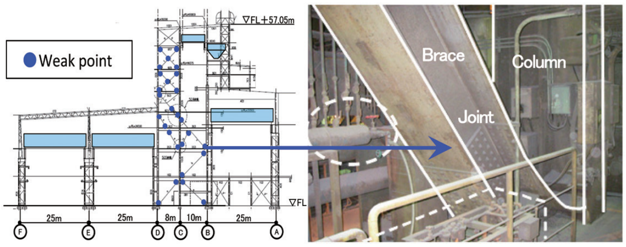


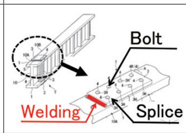
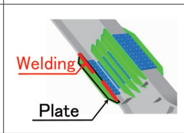
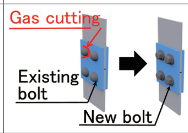
Fig. 2 Weak point in the earthquake-resistant in a steelmaking factory

Table 1 Example of a target H-shaped steel brace

Section size	Proof stress (kN)			Qut.
	Brace ①	Joint ②	Shortage ①−②	
H-900×450×50×50	27489	16640	10849	20
H-650×450×22×25	11545	8320	3225	11
H-588×300×12×20	4369	2185 *	2185	16
H-400×400×13×21	6938	3469 *	3469	18
H-250×250×9×14	2116	1058 *	1058	17

* Assumption

Table 2 Former technology applicable in joint reinforcement

Title	Welding of splice (※ cf.1)	Welding of plate (known technology)	Replacing of bolt (known technology)
Outline			
Fire	×	×	×
Space	○	△	○

and section steel, which also requires the use of an ignition source and space around the joint, is not suitable either. For the replacement of bolts, gas cutting also involving fire is used to remove the existing bolts, so this method is inappropriate as well. Since these conventional technologies involve the use of an ignition source, development of a compact reinforcement method without using fire is required.

3. Proposal of a Reinforcement Method

A schematic view of our proposed reinforcement method is shown in **Fig. 3**. This method focuses on the use of the space surrounded by the H-beam flanges and webs. Such space has existing joints and other portions with a projection and depression, with which friction joints are formed using concrete that does not require combustion and that can be easily injected to fill up a gap.

The processes that compose this method are shown in **Fig. 4**. The space surrounded by H-beam flanges and webs is filled with concrete. Cover plates are used to hold the concrete from both sides by using the tension introduced into the PC steel rods to generate frictional force at the interface between the steel and concrete.

The transfer of load in the part reinforced by this method is shown in **Fig. 5**. The load that the existing joint cannot transmit is transmitted to the cover plates via the concrete from the existing braces (H-beams) thanks to the frictional force at the interface between the steel and concrete. The compact reinforcement part that consists of concrete, reinforcing steel plates, and PC steel rods can be constructed without using an ignition source within the cross section of the existing H-beam.

Issues that need to be solved for practically using this method are to identify the mechanical performance and to confirm the feasibility. Specific technical issues are as follows.

Matters that need to be unraveled for identifying the mechanical performance

Issue (1) Load transfer mechanism from the existing steel frame to the reinforcement part

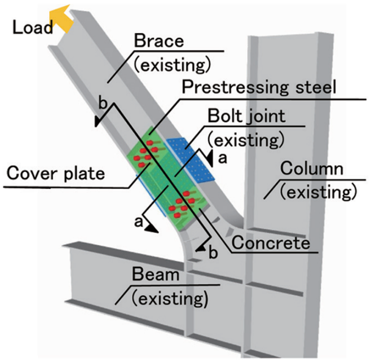


Fig. 3 Proposed reinforcement construction method

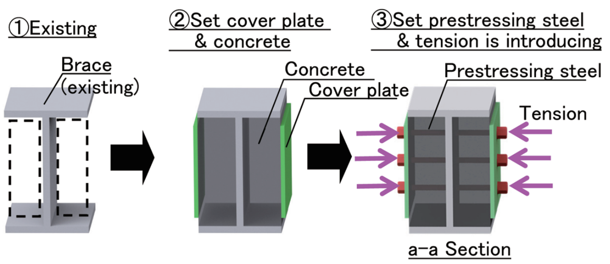


Fig. 4 Construction of the reinforcement part (a-a section)

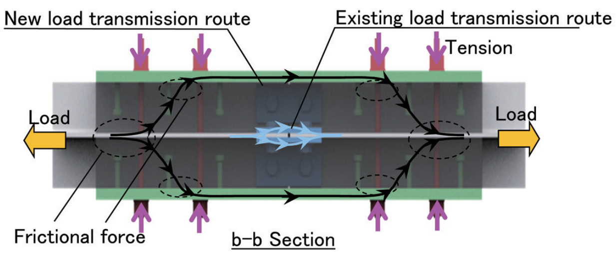


Fig. 5 Load transfer at the reinforcement part (b-b section)

Issue (2) Possibility of cumulation of individual strength of the existing joint and reinforcement part

Issue (3) Condition of stress inside the reinforcement part

Matters that need to be unraveled for confirming the feasibility

Issue (4) Efficiency of the reinforcement work

Issue (5) Influence of the creep of concrete

Issue (6) Properties of concrete as a filling

4. Identification of the Mechanical Performance

4.1 Performance confirmation tests

4.1.1 Outline of the tests

As shown in **Fig. 6**, three factors are assumed for the load transfer mechanism from the existing steel frame to the reinforcement part formed according to this method: The friction between the steel and concrete; the adherence of the concrete to steel; and the dowel effect of PC steel rods. We checked the contribution rate of each factor to the load transmitted from the existing steel frame to the reinforcement part.

The setup of the specimen is shown in **Fig. 7**. A 2000-kN jack was used to perform monotonic tensile loading to the specimen supported by pins at both ends. The standard specimen is shown in **Fig. 8**. The members that composed the standard specimen are shown in

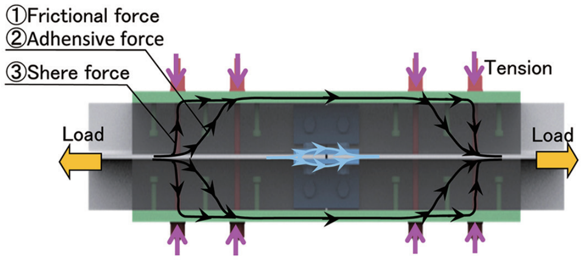


Fig. 6 Load transfer mechanism to the reinforcement part

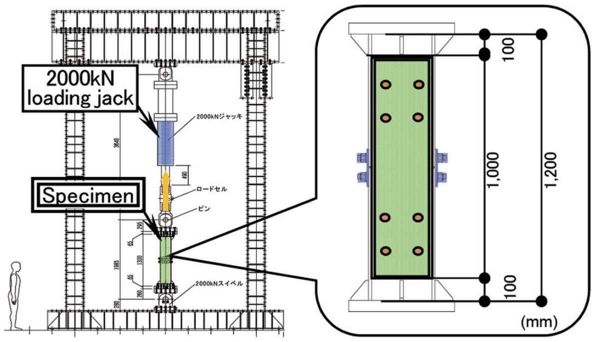


Fig. 7 Setup of specimen

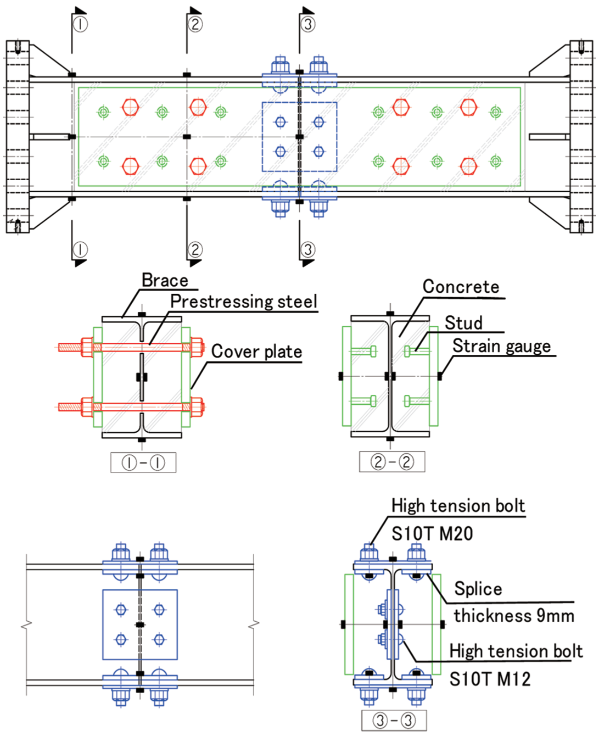


Fig. 8 Standard specimen

Table 3. As the steel frame (base material), H-244 × 175 × 7 × 11 (equivalent to the real size) was used. Stud bolts were welded to the cover plate in order to prevent the concrete from moving out of alignment. The PC steel rods were unbonded steel strands, which would not contact the concrete. The tensile force applied for each rod was 100 kN. High-strength concrete (Fc = 100 N/mm^2) using an agent for reducing shrinkage mixed with it was used. A load cell in-

Table 3 Each part of specimen

Brace	H-244×175×7×11 (SM490, SS400)			
Reinforced joint	Joint	Flange	4HTB-M20, 2PL-9 (SS400), 4PL-6 (SS400)	
		Web	4HTB-M12, 2PL-9 (SS400)	
	Reinforcement	Cover plate		t19 (SS400), Stud (D13)
		Prestressing steel		ϕ15mm (TypeC1)
			Tension	100 kN/number
		Concrete		Fc = 100 N/mm²

Table 4 Design strength of specimen

Brace (kN) ①	Joint (kN) ②	Reinforced joint (kN) ③ = ①	Reinforcement (kN) ④ = ③ − ②
1 330	700	1 330	630

Table 5 Tension test result of the steel frame brace

Materia	Yield strength (N/mm²)	Tensile strength (N/mm²)	Elongation (%)
SM490	433	543	25
SS400	311	465	29

stalled at the end of the jack was used to measure loads and displacement in the longitudinal axis direction of the specimen. Uniaxial gauges installed at the locations marked with black rectangles in Fig. 8 were used to measure the strain at each portion of the specimen.

Table 4 shows the design strength of the specimen. The assumed steel frame base material was SS400 the tensile yield strength of which was 1 330 kN. Further, the assumed strength of the existing joint was 700 kN, which was approximately half that of the steel frame base material. The specimen was designed such that high-tensile bolts would slide at 700 kN. Moreover, the strength of the reinforced joint was equal to or more than the tensile yield strength of the steel frame base material. The strength of the reinforcement part was 630 kN obtained as a result of deducting the strength of the existing joint from the strength of the reinforced joint.

During the design of the specimen, the tensile force to be applied, type of material, and diameter of the PC steel rods for each of the portions of the specimen were determined by calculating using the friction coefficient 0.5 of concrete on steel, considering the assumed required frictional force. The strength of the concrete was determined based on the diagonal tensile stress caused by the compressive force and shear force due to the tensile force applied to the PC steel rods. The type of material and cross-sectional shape of the cover plate were determined on the assumption that the strength required at the reinforcement part would be transmitted in the form of tension. It was confirmed that the stress intensity of the steel frame base material was kept within the allowable range in the portions where the tensile force of the PC steel rods was transmitted via the concrete.

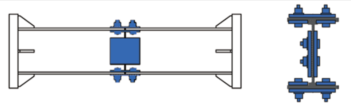
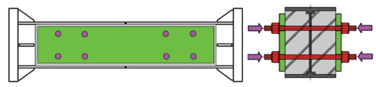
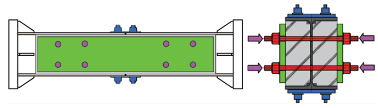
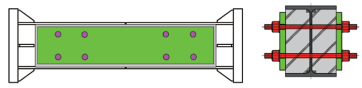
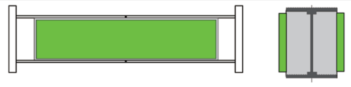
The results of the tensile test for the steel frame base material are shown in **Table 5**. The results of the material test of the concrete shown in **Table 6** indicate that the slump flow of the high-strength concrete adopted for this construction method is large at 620 mm×610 mm, which means high fluidity.

Table 7 shows the list of specimens. The specimen in Case 1

Table 6 Material test result of concrete

Materia	Compression strength (N/mm²)	Slump flow (mm)
Fc100	125	620×610

Table 7 List of specimen

Case	Outline	Joint	Brace
1		○	SS400
2		×	SS400
3		○	SM400
4		×	SS400
5		×	SS400

had only the existing joint. The one in Case 2 was the reinforcement part itself. Case 3 was a specimen after the reinforcement (the existing joint+reinforcement part). The specimen in Case 4 was the reinforcement part itself, the PC steel rods of which were not subjected to the application of tensile force. Case 4 was used to check the quantity of loads to be transmitted by adherence of the concrete to steel and the dowel effect of the PC steel rods. The specimen in Case 5 has no PC steel rods at the reinforcement part. Case 5 was to check the quantity of loads to be transmitted by adherence of the concrete to steel.

4.1.2 Test results

Issue (1): Unraveling the load transfer mechanism from the existing steel frame to the reinforcement part

The relationships between the load and displacement for Case 2, Case 4, and Case 5 are shown in **Fig. 9**. In Case 2, the load elastically increased to approximately 1 mm of displacement, and after that the stiffness decreased. Near displacement of 2 mm, the friction between the steel frame base material and concrete broke, and the sliding began. However, near displacement of 3 mm, the load started increasing again due to the PC steel rods coming into contact with the steel frame base material until the target strength of the reinforcement part was exceeded. In Case 4, as the stiffness decreased in the early stage of the loading, the loading was ended at around 150 kN. In Case 5, the load remained at approximately 10 kN until the loading was terminated.

From the above, the strength at the displacement that was where the slide occurred in Case 3 was 545 kN, which constituted the grounds for determining the design strength. The friction accounted for 84% of this 545 kN dominantly over the proportion of the other two load transfer mechanisms. The apparent frictional coefficient was 457 kN/(100 kN/surface×8 surfaces)=0.57. This confirmed that the design frictional coefficient of 0.5 is considered to be on the safe side.

Issue (2) Possibility of cumulation of individual strength of the existing joint and reinforcement part

The relationships between the load and displacement for Case 1, Case 2, and Case 3 are shown in **Fig 10**. The broken line in the figure indicates an assumed load-displacement relationship for which Case 1 and Case 2 were simply cumulated (hereinafter referred to as "simple cumulation"). In Case 2, the high-tensile bolts of the existing joint began sliding when the displacement exceeded 1 mm, so the loading was terminated. In Case 3, when sliding began at the maximum strength of 1 460 kN, exceeding both the target strength of 1 330 kN and strength of the simple cumulation, the loading was terminated. The reason why the strength of the simple cumulation was exceeded is probably because of the transmission of loads by the bearing strength of the existing joint and concrete.

The relationship between the load and displacement and that between the load and strain for Case 3 is shown in **Fig. 11**. The load of the existing joint was calculated based on the measured strain value. The load of the entire specimen was obtained by measurement using the load cell. The load of the reinforcement part was obtained by deducting the load of the existing joint from the load of the entire specimen. It was confirmed from the load-displacement relations that the maximum strength of the entire specimen is determined by the slide of the high-tensile bolts at the existing joint. Furthermore, it was confirmed that the reinforcement part can continue stably bearing the loads after the slide begins at the existing joint. In addition, the load-strain relations show that both the existing joint and reinforcement part were within each elastic range.

It was thus confirmed that when a reinforcement part is combined with an existing joint, the maximum strength exceeds the simple cumulation, so designing using a simple cumulation is considered to be on the safe side. It was also confirmed that the reinforcement part can maintain the strength after sliding begins at the existing joint.

4.1.3 Damage after the loading test

The damage of the specimen in Case 3 after the loading is shown in **Photo 2**. Many cracks can be seen around the through holes where the PC steel rods penetrate the concrete, but no crushing is evident. In addition, fine cracks are seen on the concrete at the existing joint, but no crushing is evident. Therefore, it was confirmed that the concrete did not break due to diagonal tensile stress and functioned as a splice for a high- tensile bolted friction joint.

4.2 FEM analysis

4.2.1 Outline of the analysis

Marc/Mentat, a general-purpose program, was used for the analysis. An analysis model is shown in **Fig. 12**. The specimen used in Case 3 of the performance confirmation test was converted into a half-size model attentive to the symmetry, using a solid element for

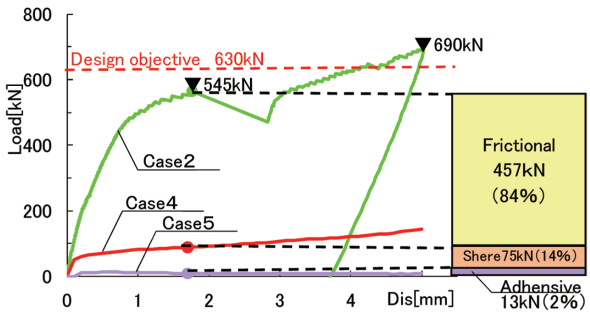


Fig. 9 Load-displacement relation (Case 2, 4, and 5)

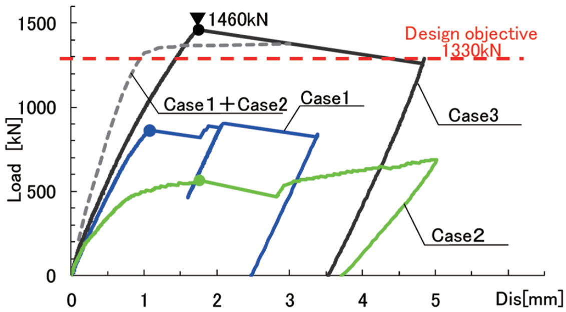


Fig. 10 Load-displacement relation (Case 1, 2, and 3)

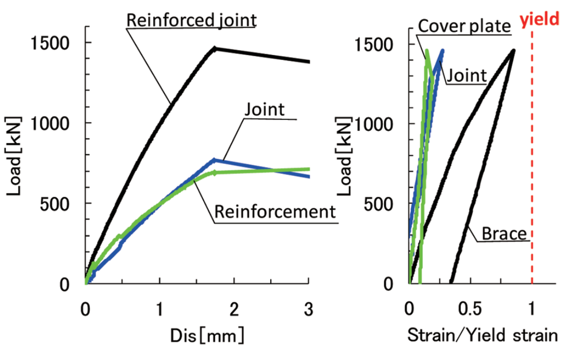


Fig. 11 Load-displacement & strain relation (Case 3)

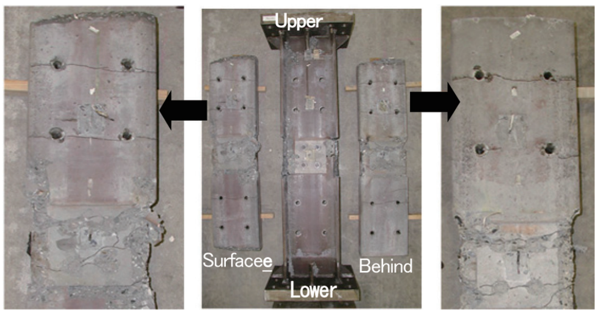


Photo 2 Damage situation after a test (Case 3)

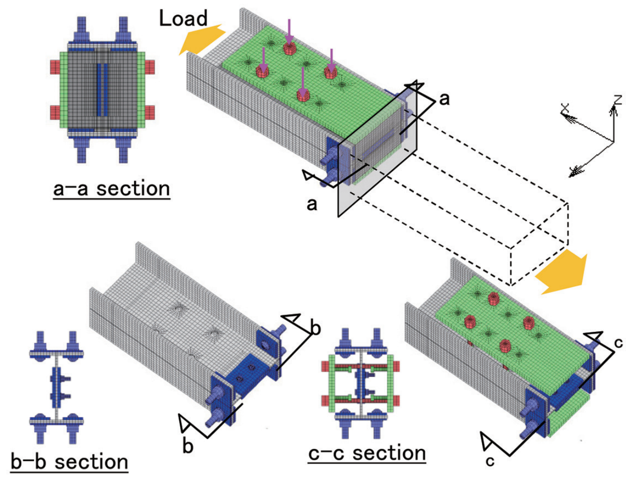
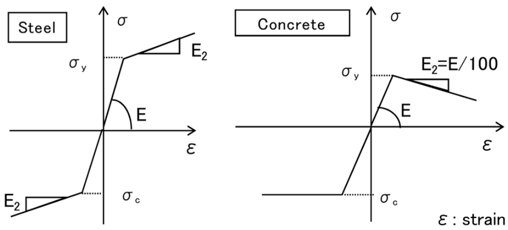


Fig. 12 FEM analysis model

Table 8 Physical properties value of the FEM analysis

		Steel						Concrete
		Brace	Joint		Cover plate		Prestressing steel	
			PL6	PL9		Stud		
Young's modulus (E)	10^3 N/mm^2	205	205	205	205	205	205	37
Yield stress (σ_y)	N/mm^2	433	190	290	275	405	1 080	8
Compression yield stress (σ_c)	N/mm^2	433	190	290	275	405	1 080	100
Poisson's ratio (v)	—	0.3	0.3	0.3	0.3	0.3	0.3	0.2



each component. In the model, the friction coefficient of the concrete on the steel frame base material was set to 0.57 as obtained from the performance confirmation test, and a spring was used. **Table 8** shows the physical properties of each component. The results of the tensile test were used for the yield stress intensity of the steel material, and the Mises yield condition was used. For the compressive strength and initial stiffness of the concrete, the results of the compressive test and the Mises yield condition were used. As the tensile strength of the concrete, 7.5 N/mm^2 was used according to a reference document on the list at the end of this paper. The Crack Date option was used to simulate the occurrence and closing of cracks on the pulling side. Simple tensile loading that was the same as that in the test was used for 3-D elasto-plastic analysis.

4.2.2 Analysis results

Issue (3) Condition of stress inside the reinforcement part

The comparison between the performance confirmation test results of the specimen in Case 3 and the analysis results is shown in **Fig. 13**. The load-displacement relations for the reinforcement part, existing joint, and reinforced joint form curves have good correlation with each other overall. Therefore, we judged that this model could be used to analyze the performance confirmation test results. The stress contour chart for the inside of the reinforcement part when the stiffness of the reinforced joint starts declining (Step 16) in Fig. 13 is shown in **Fig. 14**. The steel members were within the elastic range, which means they were in a sound condition. A small portion of the concrete filled into the clearance between the PC steel rod and the base steel frame was crushed, but the most part was within the elastic range in a sound condition. Similarly, **Fig. 15** shows the frictional force of the base steel frame and the bearing force of the existing joint at Step 16. The figure indicates that the tensile force of the PC steel rods was transmitted to the webs and flanges of the steel frame base material via the concrete, thereby causing the frictional force to occur at the webs and flanges. In addition, Fig. 15 also shows that the existing joint was in contact with the concrete, producing bearing force.

Therefore, it was confirmed that the inside of the reinforcement part remained sound, and that the loads were transmitted to the flanges of the base steel frame (H-beam) by friction.

Next, we analyzed and examined the effect of the stud bolts welded to the cover plates, which could not be checked through the performance confirmation test. Theoretically, loads can be transmitted from the concrete to the cover plates only by frictional force, so we thought that welding of stud bolts could be omitted if the load to be transmitted due to the shear key effect of the stud bolts and concrete was small.

A comparison of the affected conditions between the cover plates of Case 3 in the performance confirmation test with and without stud bolts is shown in **Fig. 16**. The line plot (a) of load-displacement relations shows that the amount of load transmitted by the reinforcement part was at the same level without stud bolts, until the load reached the maximum strength (sliding strength of the existing joint). The contour chart (b) showing the frictional force on the welded surfaces of the cover plates also shows that there was no significant difference regardless of whether stud bolts were installed.

As described above, it was confirmed that the omission of stud bolts, although installed during the performance confirmation test, had little effect on the performance of the reinforcement part for transmitting.

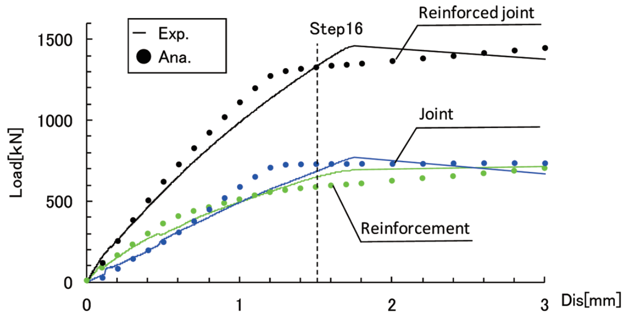


Fig. 13 Comparison of an FEM analysis and a test

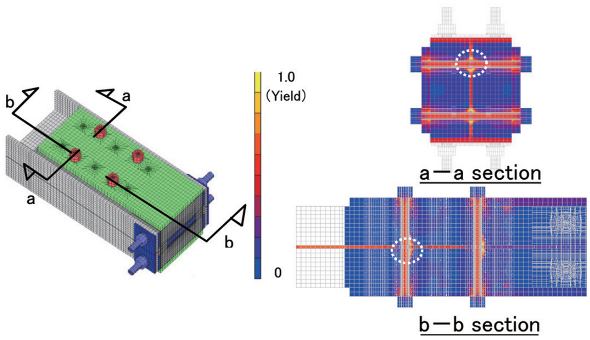


Fig. 14 Stress of a reinforcement (Step16)

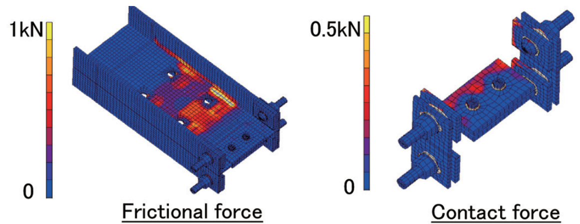
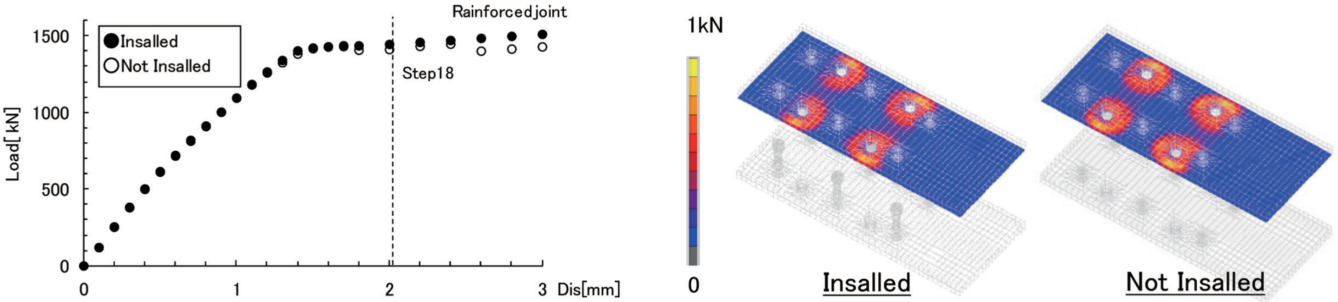


Fig. 15 Frictional force of a steel brace and the contact force of a joint



(a) Load - displacement relation (b) Frictional force of a cover plate (Step18)

Fig. 16 Comparison of a stud bolt presence

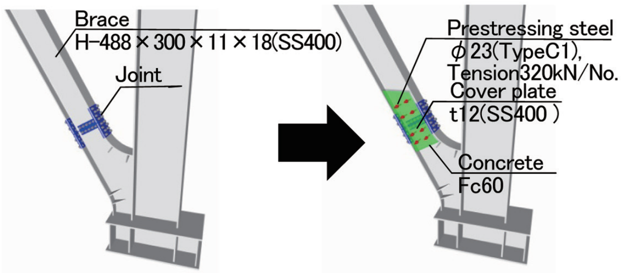


Fig. 17 Outline of an actual making test

Photo 3 Specimen after reinforcement

	Drilling	Setting cover plate	Filling concrete		Introducing tension & Touch up paint		Completion	
			Filling	Curing	Tension	Touch up	Total	vs Welding
day	0.5	0.5	0.5	28.0	1.5	1.0	31.0	+24
man day	0.5	1.5	2.0		1.5	2	7.5	−70%

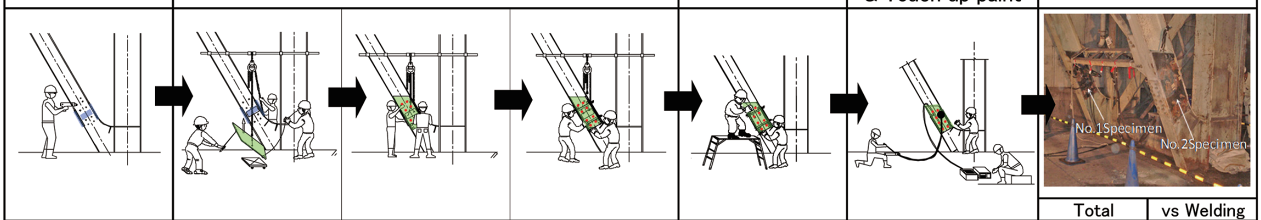


Fig. 18 Procedure of making

5. Confirmation of the Feasibility

5.1 Real size production test

5.1.1 Outline of the test

Some H-beam braces in an unused building of Nippon Steel & Sumitomo Metal were used for the test in which the proposed reinforcement parts were produced, measured, and removed. A schematic view of one of the specimens is shown in **Fig. 17**. The appearance of the specimens is shown in **Photo 3**. The size of the H-beams (base steel frames) was H-488×300×11×18 (SS400). High-tensile bolts were used as the existing joint fasteners. We conducted trial design for these existing joints such that the reinforcement parts would not be damaged until the base steel frames yielded in tension. As a result, eight PC steel rods (C-1) with a diameter of 23 mm were used. The tensile force was 320 kN per rod. For the concrete, Fc=60 N/mm^2. The thickness of the cover plate was 12 mm (SS400).

5.1.2 Test results

Issue (4) Efficiency of the reinforcement work

The procedures of reinforcement along with the confirmation results of man-hours are shown in **Fig. 18**. It was confirmed that the reinforcement part can be constructed in accordance with the construction procedures that we developed. Furthermore, it was estimated that the total man-hours for the entire processes including the drilling, applying tensile force, and touch-up (painting) would be approximately 70% of those taken by the conventional reinforcement involving welding steel plates. However, the total construction period will be longer than that of the conventional method, as it takes time for concrete to cure.

Issue (5) Influence of the creep of concrete

The changes of the tensile force applied to PC steel rods over time are shown in **Fig. 19**. The changes in the tensile force according to the creep of concrete were measured with strain gauges installed onto the PC steel rods for six weeks. The rate of effective-

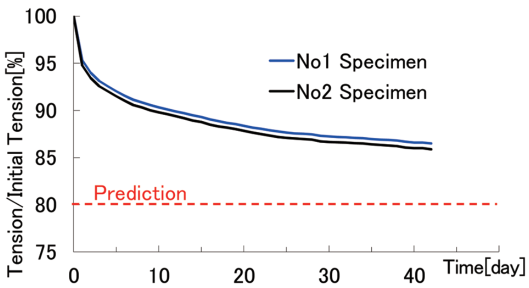


Fig. 19 Aging changes of the tension of the prestressing steel

Photo 4 Filling situation of the concrete

ness (the force initially applied − force reduced) remained at approximately 85%. It was confirmed that the rough predicted value of 80% according to reference 3) is considered to be on the safe side.

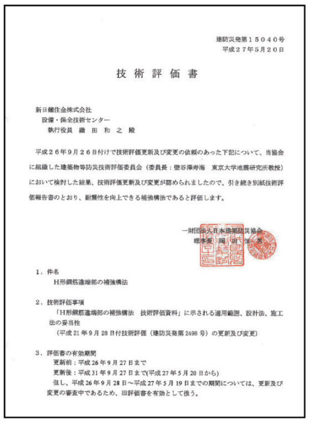
Issue (6) Properties of concrete as a filling

The condition of the concrete after the reinforcement part was removed is shown in **Photo 4**. The proposed reinforcement method uses high-strength concrete with high fluidity. It was confirmed that space with projections and depressions (e.g., bolts of existing joints) can be filled up with such concrete in a solid way.

6. Conclusion

In this paper, we propose a reinforcement method for the joints of H-beam braces used in steelmaking plants. This method uses a compact composite structure of steel and concrete, requiring no ignition source. We confirmed the mechanical performance and feasibility through the experiments and analysis. We report the following six matters.

(i) Friction is the prevailing mechanism for transmitting loads to the reinforcement part. A design friction coefficient of 0.5 is considered to be on the safe side.
(ii) The strength of an existing joint and a reinforcement part can be simply cumulated.
(iii) Frictional force occurs at both the webs and flanges at the interface between the base steel frame (H-beam) and concrete.
(iv) The man-hours required for the reinforcement can be reduced compared to those required for the conventional reinforcement method in which steel plates are welded.
(v) For the rate of effectiveness of the tensile force applied to PC steel rods that changes according to the creep of the concrete, 80% is considered to be on the safe side.
(vi) High-strength concrete can be used as a solid filling.


建防災発第１５０４０号
平成２７年５月２０日

技 術 評 価 書

新日鐵住金株式会社
設備・保全技術センター
執行役員 織 田 和 之 殿

平成２６年９月２６日付けで技術評価更新及び変更の依頼のあった下記について、当協会に組織した建築物等防災技術評価委員会（委員長：壁谷澤寿海 東京大学地震研究所教授）において検討した結果、技術評価更新及び変更が認められましたので、引き続き別紙技術評価報告書のとおり、耐震性を向上できる補強構法であると評価します。

一般財団法人日本建築防災協会
理事長 岡 田 恒 男

１．件名
H形鋼筋違端部の補強構法

２．技術評価事項
「H形鋼筋違端部の補強構法 技術評価資料」に示される適用範囲、設計法、施工法の妥当性
（平成21年9月28日付技術評価（建防災発第2498号）の更新及び変更）

３．評価書の有効期間
更新前：平成26年9月27日まで
更新後：平成31年9月27日まで(平成27年5月20日から)
但し、平成26年9月28日〜平成27年5月19日までの期間については、更新及び変更の審査中であるため、旧評価書を有効として扱う。

Fig. 20 Technological evaluation document

	No.2 Steelmaking Plant of Kimitsu	No.1 Steelmaking Plant of Kimitsu
Photo		
No.	2	77

Photo 5 Application results

Based on the achievement described above, we created design and construction guidelines, and obtained a technical assessment certificate for the method from the Japan Building Disaster Prevention Association (**Fig. 20**). After that, we actually applied the method to two steelmaking plants (**Photo 5**). We will strive to further expand the application of this method going forward.

Acknowledgments

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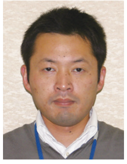

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