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EXPERIMENTAL STUDIES OF TOWER STRUCTURES WITH HYSTERETIC DAMPERS

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SUMMARY

The authors propose the concept of damage-tolerant truss-structures by introducing hysteretic dampers into them, which dissipate the seismic energy and avoid the buckling of other members by limiting the response forces. This concept is employed in practical communication towers and its validity is confirmed. This concept simplifies reinforcement work since only a few diagonal members need to be replaced with damper members.

In this study, the actual sizes of the structures around the dampers have been mocked up, and cyclic loadings up to 4% of the story drift angle are performed. These results are compared with those of the diagonal members made of normal pipes and concrete in-filled members; further, the energy dissipation capacity of each system is discussed. When the diagonal members are replaced with damper members, site work required for connections become minimum and simple. In this study, various types of connections have been tested, and their out-of-plane stabilities have been compared and discussed.

Keywords: *Truss Tower, Seismic Retrofit, Buckling Restrained Brace, Connections*

1. INTRODUCTION

Microwave communication towers owned by electric companies have been placed on the top of buildings in the city and suburban areas; they are used to communicate information regarding the control of power plants providing electric power within the coverage area. These high-rise communication towers consist of steel truss structures with pipe sections. Thus far, these structures have been basically designed to withstand wind forces, however, it was suggested that such structures undergo serious damages when subjected to large seismic forces, where amplitude fluctuations are caused due to the building structures on which the towers are installed. If the truss members collapse because of the seismic forces, truss structures with less-ductile characteristics would collapse due to the buckling of members. Normal reinforcement for strengthening the weak members would not necessarily be sufficient, because the other members or connections might become critical after the weak members are reinforced: further, this would necessitate the reinforcement of all the members. Such reinforcements would reduce the own period of the structure, often resulting in higher seismic inputs. Therefore, normal reinforcement can prove to be time consuming and expensive.

The authors have proposed a seismic retrofit method in which the critical truss members are replaced with buckling restrained braces (BRB), which function as high-performance hysteric dampers, and proved their effects by means of analyses [1][2]. On the basis of these studies, we mock up the actual sizes of the truss frames around

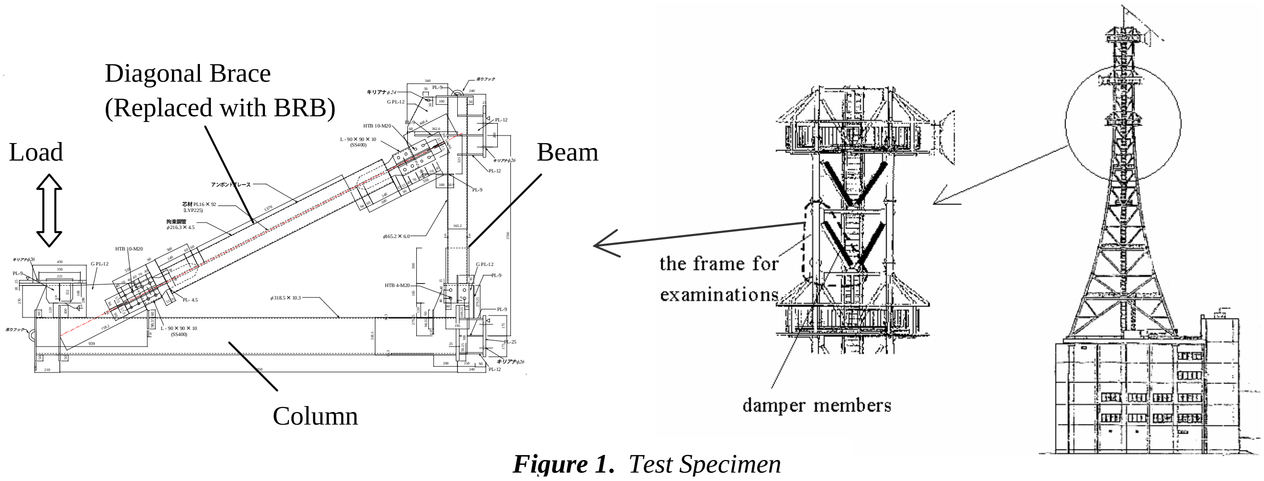


Figure 1. *Test Specimen*

the dampers, and perform hysteresis-loading tests. The results are compared with those of diagonal members made of normal pipes or concrete in-filed members: further, the energy dissipation capacities of each system are discussed. When these structures are reinforced, the amount of retrofit work needed for connections become minimum and simple. In this study, two types of connections are tested, and their out-of-plane stabilities are compared and discussed. The application layout of BRB for communication towers and mock ups in the test specimen are shown in Figure 1.

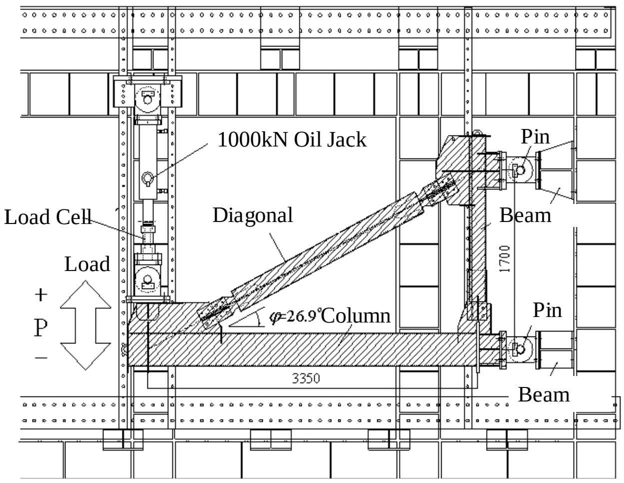


Figure 2. *Test Configuration*

2. EXPERIMENTAL METHOD

Real-size mock-up tests are carried out for the frame around the critical positions shown in Figure 2. A list of the specimens is summarized in Table 1. The specimens consist of four types of designs; (1)TO denotes a normal pipe member, (2)TC denotes a concrete in-filled member and (3)TA and TB denote BRBs. In the BRB reinforcement design, the diagonal members are replaced with BRBs, whose yield strength are equivalent with the buckling strength of the original diagonal pipe members. In addition, the two connection types shown in Figure 3 are tested. In one type, additional stiffener plates are welded onto gusset plates to prevent buckling along the out-of-plane direction of gusset plates (TB): in the other type, angle plates are used instead of splice plates (TA). The connection of angle plates is simpler and no welded plates are required; therefore, site work required at higher locations is considerably convenient than the use of on-site welding plates.

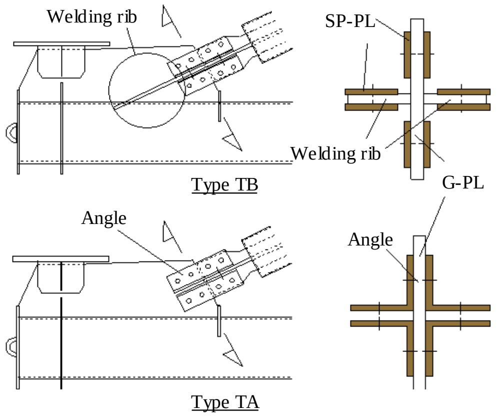


Figure 3. *Types of Connections*

Figure 4 shows the loading program. The basic loading history consists of increasing cyclic loading up to 1/25 story drift. In the near field loading histories of TA-1’ and TA-2’, the structure is assumed to be directly above the epicentre of an earthquake, and story drifts of 1/20 - 1/50 are modelled.

Table 1. *Diagonal Members*

Test piece	Brace	Bolt	Type	Loading History
TA-1	BRB-16x92(Py=300kN)	4M-20	Angle joint	Basic
TA-1'				Near-Field(tension)
TB-1			Welding rib	Basic
TC-1	pipe ф165.2x4.5	4M-20	Concrete in-filled	
TO-1			Ordinary	
TA-2	BRB-16x58(Py=190kN)	2M-20	Angle joint	
TA-2'				Near-Field(compression)

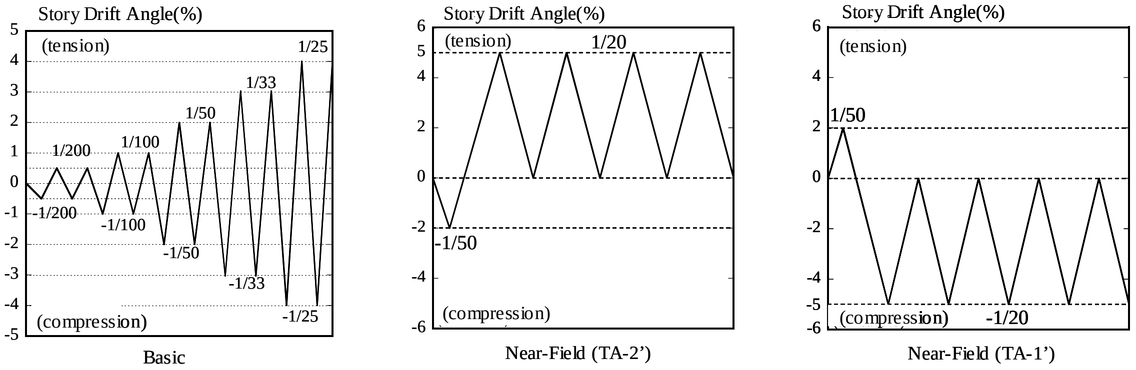


Figure 4. *Loading Program*

3. EXPERIMENTAL RESULTS REGARDING THE CHARACTERISTICS OF DIAGONAL MEMBERS

The test results are shown in Figure 5-8. In each figure, part (a) shows the load-displacement relationships of the truss structures, and part (b) shows the axial force-deformation relationships of the diagonal members. For TO-1, a story drift of up to 1/100 was followed by slipping of the connection bolts, the diagonal members then started to buckle at the first 1/50 of the compression cycle. Elbow buckling occurred at the second 1/50 of the compression cycle, and the member was torn off at the first 1/25 of the tension cycle. The maximum axial force to initiate buckling was approximately 820kN.

For TC-1, the beam side gusset plate started to deform along the out-of-plane direction at the first 1/50 of the compression cycle, and the gusset plate was completely buckled at the first 1/33 of the compression cycle before the brace buckled. The concrete in-filled member almost did not dissipate energy. On the basis of these results, it is found that the strengthening diagonal members might collapse the other parts as connections.

For the types with BRBs, braces and connections did not buckle, and they exhibited fairly stable and symmetrical hysteresis loops up to a story drift of 1/25. Each specimen dissipated sufficient energy until the core plate fractured (For TB-1, the core plate fractured at the third 1/25 of the tension cycle; for TA-1, the fourth 1/25 of the tension cycle; and for TA-2, the fourth 1/25 of the tension cycle).

For Type TA-1 and 2 with angle joints, the out-of-plane deformation did not increase and showed stable deformation capacity, similar to that of TB. The connection and main frame were not damaged until the BRB exhibited sufficient plastic deformation capacity and the connection damage was avoided as compared to TO-1 and TC-1.

The test results for near-field loading programs are shown in Figure 9 and 10. In each figure, part (a) shows the load to vertical displacement relationships of the truss structure and part (b) shows the axial force to axial deformation relationships of the diagonal members. For TA-1' and TA-2', in spite of comparatively severe story drifts, no deformation and buckling occurred at the braces and connections. Both these types of connections exhibited stable hysteresis loops, at the 10th cycle in TA-1' and 16th cycles in TA-2'. As the maximum story displacement increases, the number of hysteresis loops increases until the core plate fractures; the cumulative deformation capacity depends on the amplitude rather than the maximum displacement.

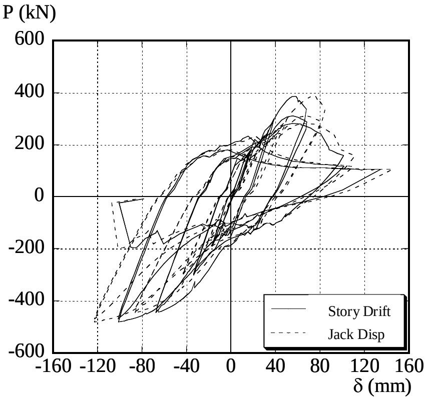


Figure 5*(a).* *TO-1 Frame*

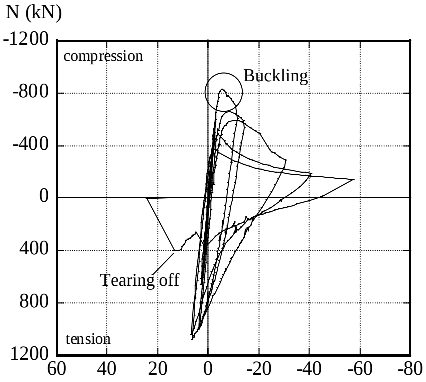


Figure 5*(b).* *TO-1 Diagondl*

Figure 5*(c).* *Diagonal Buckling inTO-1*

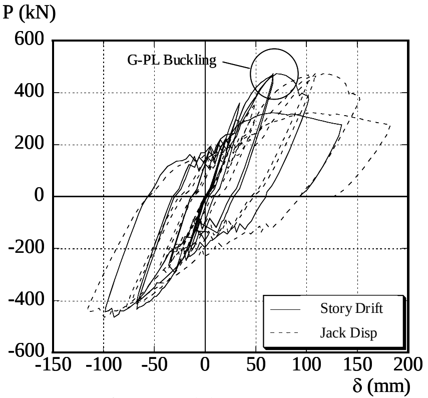


Figure 6*(a).* *TC-1 Frame*

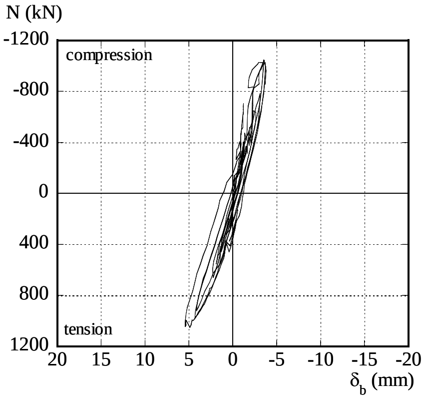


Figure 6*(b).* *TC-1 Diagonal*

Figure 5*(c).* *Gusset Pl. Buckling inTC-1*

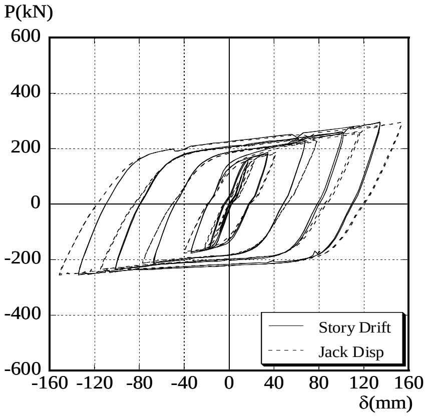


Figure 7*(a).* *TA-1 Frame*

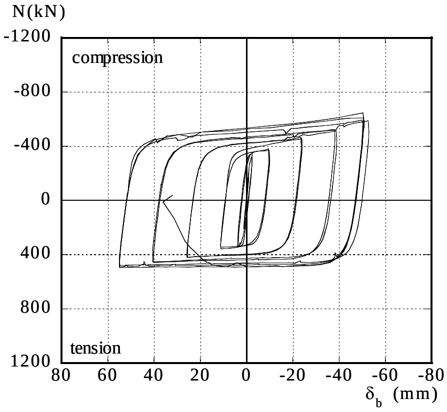


Figure 7*(b).* *TA-1 Diagonal*

Figure 7*(c).* *At the End of the Test (TA-1)*

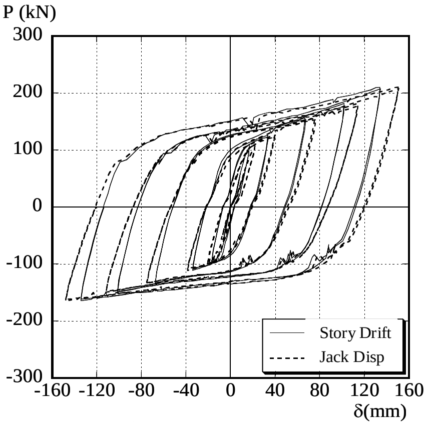


Figure 8*(a).* *TA-2 Frame*

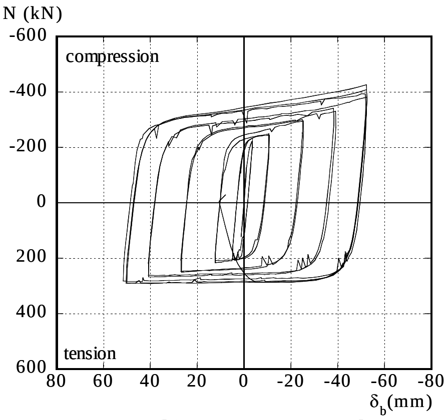


Figure 8*(b).* *TA-2 Diagonal*

Figure 8*(c).* *At the End of the Test (TA-2)*

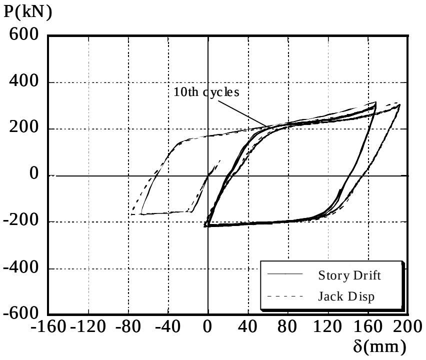


Figure 9*(a).* *TA-1' Frame*

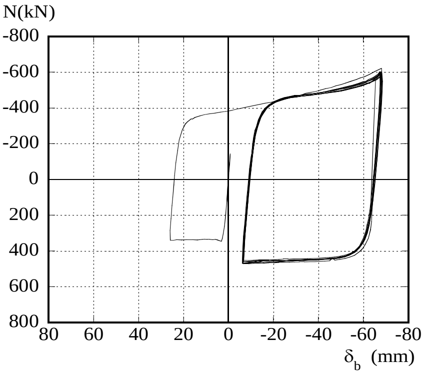


Figure 9*(b).* *TA-1' Diagonal*

Figure 9*(c).* *At the End of the Test(TA-1')*

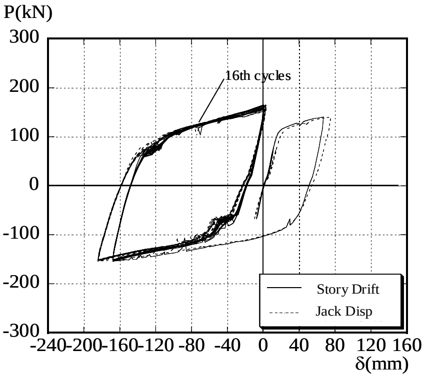


Figure 10*(a).* *TA-2' Frame*

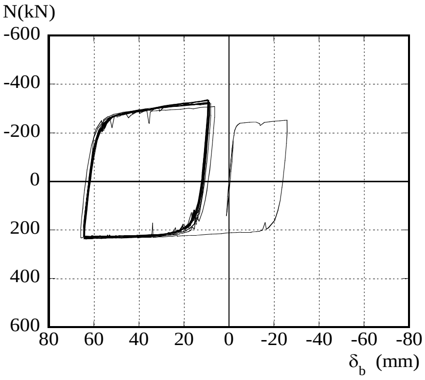


Figure 10*(b).* *TA-2' Diagonal*

Figure 10*(c).* *At the End of the Test(TA-2')*

4. BEHAVIOR OF THE CONNECTION

Figure 11 shows the strain values of the gusset plates. After the brace buckles under compression, the gusset plate of TO-1 was considerably plasticized before the maximum strength of the braces under tension was attained. The gusset plate of TC-1 buckled along the out-of-plane direction and was mostly plasticized before the brace buckles under compression. In Type TB-1, the strain value of the gusset plate is maintained within the elastic range because the maximum strength of the brace is limited. The strain value of the gusset plate of TA-1 is greater than that of TB-1 because it does not have welded ribs, however, it is still almost elastic. In TA-2, a similar result was obtained as that of TA-1.

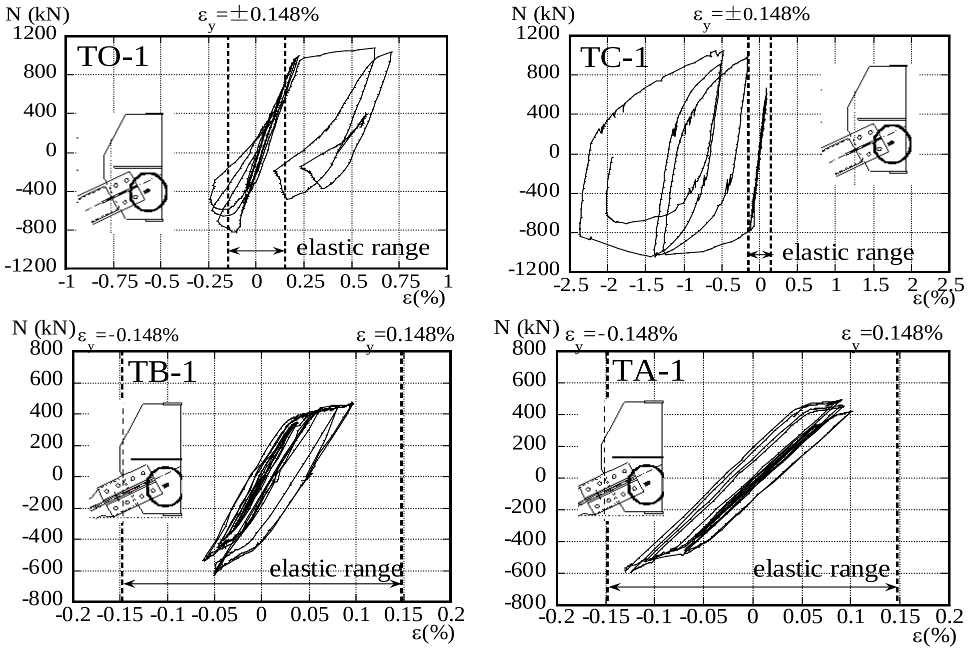


Figure 11. *Strain Values of the Gusset Plates*

Table 2. *Deformations of the Bolt Holes*

TO-1

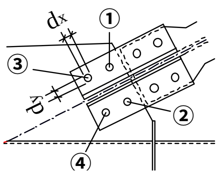
	d_x dir.	d_y dir.
1	**26.55**	22.05
2	**26.20**	22.35
3	**27.75**	22.35
4	**26.75**	22.90
5	**26.05**	22.05
6	**25.70**	22.10
7	**27.75**	22.50
8	**26.95**	22.80
9	22.10	22.10
10	22.15	22.15
11	22.05	22.00
12	22.40	22.00

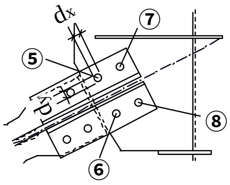
TC-1

	d_x dir.	d_y dir.
1	**27.80**	23.05
2	**26.60**	22.20
3	**28.10**	22.35
4	**26.40**	22.20
5	**28.30**	22.80
6	**28.90**	22.25
7	**27.80**	22.75
8	**27.65**	22.10
9	21.95	21.95
10	21.95	22.05
11	22.00	21.95
12	22.05	21.90

TA-1

	d_x dir.	d_y dir.
1	21.95	21.95
2	22.30	21.90
3	22.00	21.95
4	22.05	22.05
5	22.20	22.10
6	22.35	22.35
7	22.20	22.10
8	22.15	22.35
9	22.50	22.10
10	22.25	22.25
11	22.40	21.90
12	22.30	21.95





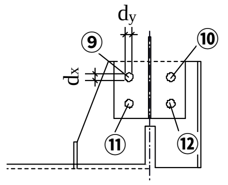


Table 2 lists the displacement values of bolt hole in the gusset plates after the test was conducted. In TO-1 and TC-1, the bolt holes were deformed due to the bearings because the brace strength is greater than the slipping strength of the joints and bolts. In particular, the hole deformation in TC-1 is remarkable. In Type TA-1, the deformation of the bolt holes was not observed because the brace strength is limited.

5. BEHAVIOR OF THE BRB ENDS

In order to secure the energy dissipation capacity of the BRB members, it is necessary to avoid out-of-plane buckling. When the length of the BRB ends with stiffener ribs is short, it induces a marginal restriction and its rotation can easily occur [3]. Then three hinges are created in the brace member, which buckles in the out-of-plane direction.

Figure 12 shows the flexural behavior model of the BRB end, and Figure 13 shows the horizontal rotation angle and axial displacement relationship. In these tests, the length of the BRB end is 300mm in order to avoid the formation of hinges. As shown in Figure 12, the BRB end deform crushing the unbonded material along the rotation. In the case that the restrained tubes do not deform, a rotational angle (1/75) is expected to occur along the thickness of the unbonded material. In these tests, it is found that all the test pieces rotate until a maximum amplitude of 1/50. Some deformation in the restrained concrete seems to increase the rotation angle; however, the rotation angle along the out-of-plane directions is kept within 1/50. Therefore, it can be concluded that these details have sufficient strength and stiffness to avoid buckling.

6. CUMULATIVE DISTORTION AND ENERGY

Table 3 and Figure 14 show the cumulative strain capacity and Table 4 and Figure 15 show the dissipated energy until the occurrence of fracture. The cumulative equivalent strain capacities in a

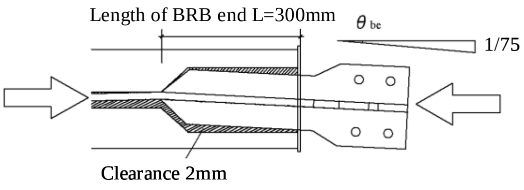


Figure 12. *Flexural Behavior at the BRB End*

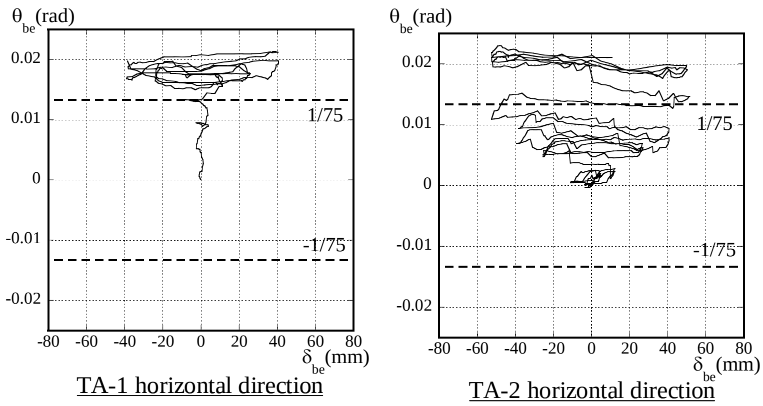


Figure 13. *Axial Deformation and Rotation Angle at the End*

Table 3. *Cumulative Strain Capacity*

Test Piece	Cumulative Strain Capacity Σ｜⊿ε｜ tension	compression	total	Ratio tension/compression	Average of Strain Amplitude ε_{tave}(%)
TB-1	44.13	42.87	87.00	1.03	4.22
TA-1	50.40	50.22	100.62	1.00	4.35
TA-1'	51.93	49.11	101.04	1.06	4.09
TA-2	42.30	43.47	85.77	0.97	3.62
TA-2'	58.92	61.09	120.01	0.96	3.55
TO-1	7.43	8.96	16.39	0.83	
TO-1(before buckling)	0.83	0.94	1.77	0.89	
TC-1	2.19	1.68	3.87	1.31	

Table 4. *Dissipated Energy*

Test Piece	Dissipated Energy Σ *Wp* (N･m) tension	compression	total	Ratio tension/compression
TB-1	215051	228010	443061	0.94
TA-1	286507	272928	559435	1.05
TA-1'	290810	273885	564695	1.06
TA-2	163507	182298	345805	0.90
TA-2'	188754	259118	447872	0.73
TO-1	46612	51405	98017	0.91
TO-1(before buckling)	90	1196	1286	0.08

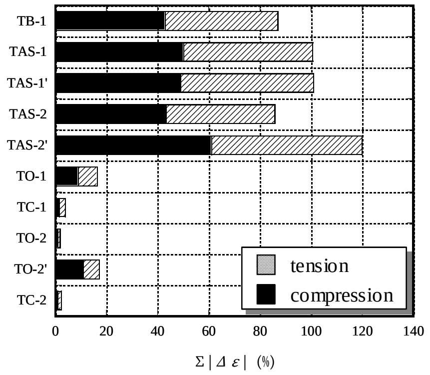


Figure 14. *Cumulative Strain Capacity*

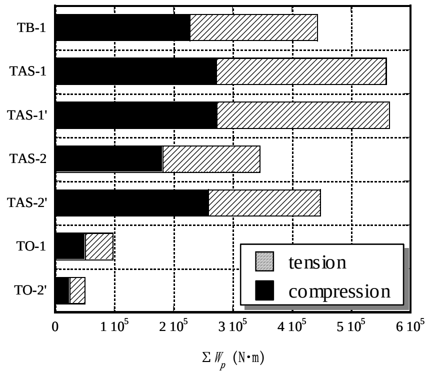


Figure 15. *Dissipated Energy*

diagonal BRB member are approximately 5 times greater than ordinary pipe members (TO-1). By estimating the performance before buckling, a BRB member has 10 times more capacity than TO-1. For the present ordinary pipe, the dissipated energy capacity is limited and the buckling possibility of the brace or connection buckling is high. By replacing the brace with BRB member, the yield strength gets limited, and damage to the other members or connections is avoided. With the near-field loading histories of TA-1' and TA-2', similar displacement capacities can be observed as the basic loading histories. Here, the damper members exhibit stable capacities in all the types of loadings.

7. STRENGTH EVALUATION OF JOINTS

Figure 16 shows a comparison of the strengths of braces and joints. At the time of designing, estimated brace strength is considerably smaller than the joint strength, however, the actual brace strength is almost equal or higher than the joint strength. Figure 17 shows the distribution map of the yield strength of steel materials. The yield strength of the pipe materials (STK400 and 490) shows considerably higher values than the design yield stress (F-value) and the distributions of ordinary steel materials (SS400 and SM490). Therefore, it is necessary to confirm that the brace strength is not higher than the joint strength.

8. CONCLUSIONS

In this paper, the seismic retrofit of communication tower structures has been proposed, where the critical members are replaced to damper members. The actual sizes of structures around the dampers are mocked up, and cyclic-loading tests have been performed. These results are compared with those of the diagonal members made of normal pipes or

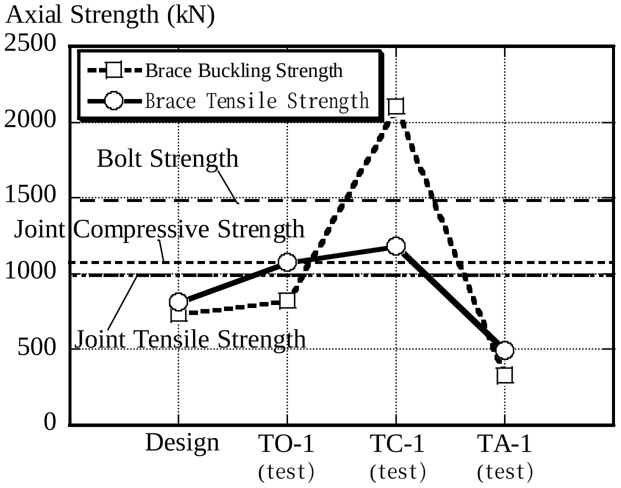


Figure 16. *Strength Comparisons*

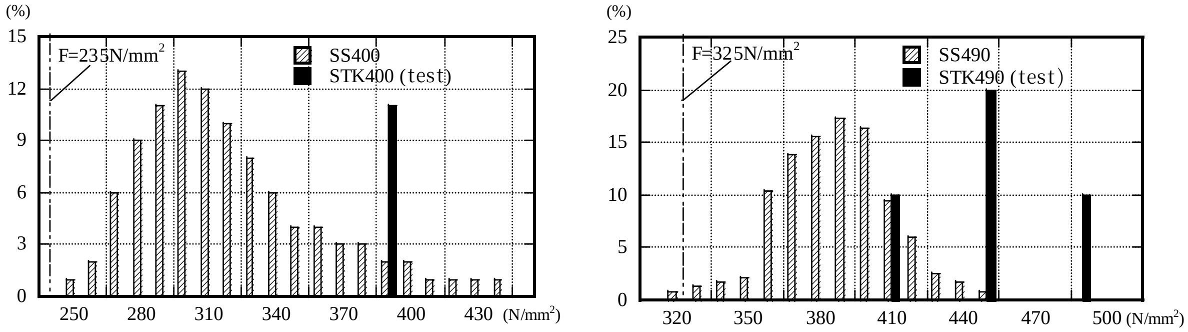


Figure 17. *Distribution Map of the Yield Strength of Steel Materials*

concrete in-filled pipes; further the energy dissipation capacities of each system were confirmed. Further, various types of connections has been tested, and their out-of-plane stabilities have been compared and confirmed. The followings conclusions have been drawn.

1) Ordinary pipe members buckle when subjected to compression side and get torn off during very early stages because of local buckling. Further under tension, the yield-stress is considerably greater than design yield stress (F-value), and the connections are likely to yield. Therefore, energy dissipation can not be expected in normal pipe members.

2) Reinforcement with concrete in-filled members is not necessarily effective. Even when the compression strength improves, the connections may become critical. Therefore energy dissipation also can not be expected.

3) When the critical members are replaced with BRBs, the experiments confirm stable hysteresis loops, displacement capacity and dissipated energy capacity. The connections and main frames were marginally damaged because the maximum strength of a BRB is limited. Therefore, reinforcement with BRBs is found to be an efficient method for the seismic retrofit of tower structures.

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