THE ULTIMATE STRENGTH OF A-SHAPED BRIDGE TOWERS UNDER CYCLIC LOADING

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1. Introduction

The purpose of this study is to create an analytical model of A-shaped bridge tower under cyclic loading in plane perpendicular to the bridge axis, to compare with previous test results, and to apply the analytical model for application of new steel types. The tests of two model towers are used. The SHAPE-1 is a prototype of Meiko-Nishi Cable-Stayed Bridge, which has center span length of 405 meter. The tower has an A-shaped with straight legs until the base. For this type, four models of 1/30 scale with different member's section are considered, two of them have bracing [1]. The SHAPE-2 is prototype of Konohana Suspension Bridge, which has center span length of 300 meter. The tower has A-shaped with inward leaning legs below the stiffening girder. Three models of 1/20 scale are considered [2]. For comparison purpose, one model of portal frame tower (SHAPE-3) is also considered. All types of model towers are shown in Fig. 1.

2. Outline of the Tests

Constant vertical load P_{const} equal to 27% of the yield load was applied on the top of the model towers, and a horizontal cyclic loading was applied at the stiffening girder level. History of load-displacement and load- strain relationships, as well as maximum horizontal load, were experimentally investigated.

3. Outline of Numerical Analysis

Ultimate strength and load-displacement relationships of model towers under the same loading conditions of the tests are computed numerically using the finite elements method program MARC. The geometrical and material nonlinearities are considered in the calculation. The in-plane failure occurs in the overall of structure. The numerical results are compared with experimental results. The comparison of the numerical calculations with experimental results are shown in Table 1. The yield displacement δ_{yo} and the yield load H_{yo} are defined as displacement and load when the first yielding occurs in the structure. Figure 2 shows an example of the results for one of the model tower (SHAPE-1 B).

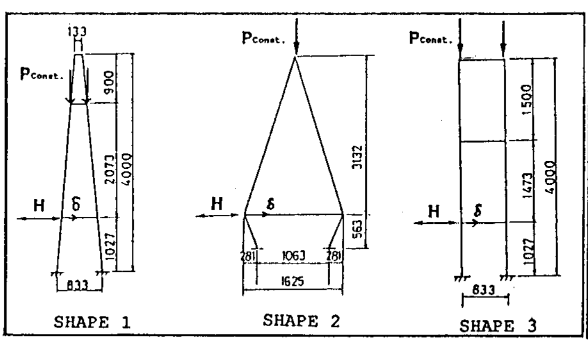

Fig.1 - Types of Model Towers

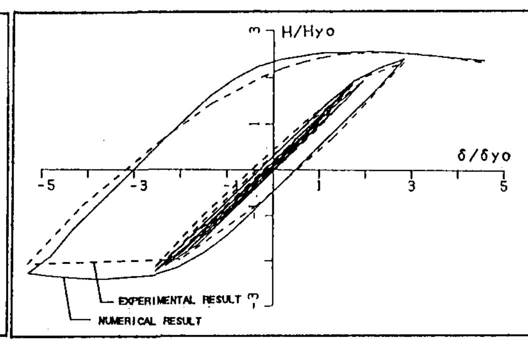

Fig.2 - The Comparison Between Experimental and Numerical Result of SHAPE-1 B Model Tower

Table 1 Comparison of Numerical Calculations with Experimental Results

Tower Type	Column	Beam	Bracing	Calculation				Experimental	(1)/(2)
				δ_{yo} (mm)	H_{yo} (ton)	H_{max} Monotonic (ton)	H_{max} cyclic (ton) (1)	H_{max} cyclic (ton) (2)	
SHAPE-1 A	H100x100x6x8	H100x100x6x8	-	4.81	2.55	5.07	5.08	5.20	0.977
SHAPE-1 B	H150x75x5x7	H150x75x5x7	-	4.55	0.91	2.18	2.33	2.39	0.977
SHAPE-1 C	H100x100x6x8	H100x100x6x8	H100x50x6x8	0.90	9.64	34.58	32.84	32.10	1.023
SHAPE-1 D	H100x100x6x8	H100x100x6x8	H100x50x6x8	0.90	9.82	35.19	33.44	33.12	1.010
SHAPE-2 A	H125x125x6.5x9	H125x125x6.5x9	-	1.54	3.83	13.42	13.52	-	-
SHAPE-2 B	H125x125x6.5x9	H125x60x6x8	-	1.74	2.74	9.84	12.37	14.20	0.871
SHAPE-2 C	H150x150x7x10	H150x150x7x10	-	1.09	4.29	19.37	21.55	-	-
SHAPE-3	H100x100x6x8	H100x100x6x8	-	5.85	2.14	4.19	4.34	4.76	0.912

4. Application of New Steel Types

Application of new types of earthquake resisting steel [3] to the analytical model is studied numerically. The first type is high-tensile strength steel with low yield ratio and the second one is high-tensile strength steel with high yield ratio, as shown in Fig. 3. Considering numerical results, ductility of different shapes of tower under cyclic loading with each type of steel is studied and also the influences of each type of new steel are investigated. The ductility ratio μ is defined as the ratio of the refence displacement δ_{90} of the structure in the inelastic range to the yield displacement δ_{yo}, that is : $\mu = \frac{\delta 90}{\delta yo}$. δ_{90} is measured from envelope curve after loading has decreased until 90 % of H_{max}. The comparison results of both types of steel are shown in Table 2. Figure 4 and 5 show comparison results of different shape of towers and different types of steel.

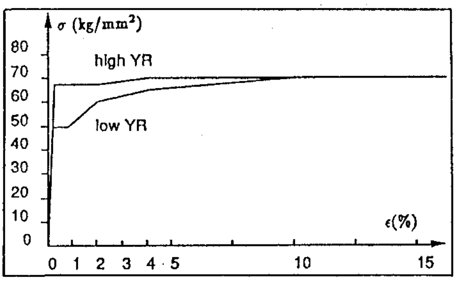

Fig.3 - Stress-Strain Curve for New Steels

Table 2 Numerical Result Using New Steels

Tower Type	Low YR σ= 50 kg/mm²				High YR σ= 68 kg/mm²			
	δ_{yo} (mm)	H_{yo} (ton)	μ	$\frac{H_{max}}{H_{yo}}$	δ_{yo} (mm)	H_{yo} (ton)	μ	$\frac{H_{max}}{H_{yo}}$
SHAPE-1 A	7.42	3.40	13.0	2.66	10.30	4.53	5.2	1.90
SHAPE-1 B	9.00	1.39	5.0	2.15	12.65	1.67	3.4	1.74
SHAPE-1 C	1.43	14.33	21.0	8.56	1.91	19.07	14.0	4.25
SHAPE-1 D	1.43	14.57	21.0	8.42	1.91	19.39	15.0	4.37
SHAPE-2 A	2.14	4.77	19.0	7.28	2.97	6.22	10.0	3.31
SHAPE-2 B	2.53	3.38	12.0	5.29	3.54	4.15	6.4	3.08
SHAPE-2 C	1.65	5.75	27.0	11.63	2.27	7.66	15.0	4.10
SHAPE-3	9.03	2.89	10.0	2.42	12.24	3.59	4.4	1.88

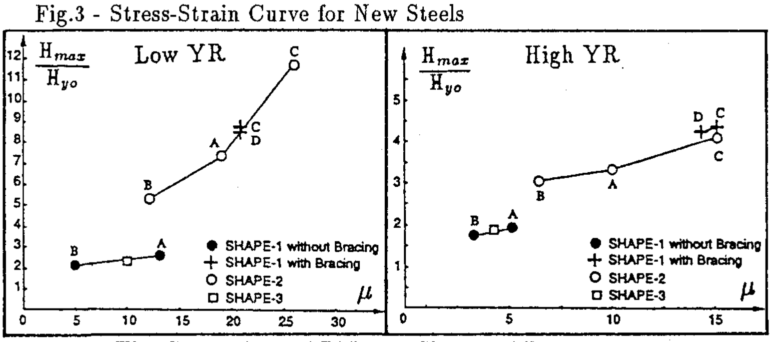

Fig.4 - The Comparison of Different Shapes of Towers

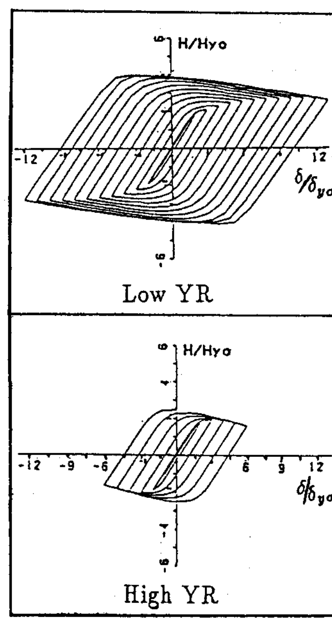

Fig.5 - The Comparison of Result of SHAPE-1 A with Different types of Steels

5. Conclusion

In this study the results from analytical model have good agreement with test results. Figure 4 shows that SHAPE-2 model towers have higher H_{max}/H_{yo} ratio than the others shapes. Figure 4 and 5 show that if high-tensile strength steel is used, the tower with low yield ratio steel has more reserve capacity and more capability to deform in inelastic range in order to resist overload compared to tower with high yield ratio steel.

6. References

[1] Y. Fukumoto, Y. Itoh, M. Katsuya: "Theoretical and Experimental studies on In-plane Strength of Towers of the Meikonishi-Ohashi Cable-Stayed Bridge", NUCE Research Report No: 8101.

[2] Y. Fukumoto, T. Usami, Y. Itoh: "Experimental Studies on the Strength of Framed Tower of Hokko-Renraku Bridge", NUCE Research Report No: 8201.

[3] Y. Simura, H. Kuwamura:"Experiment on High-Strength Steel Beam with Different Yield Ratios", Transaction of The Architectural Institute of Japan, 1987.