PREDICTION OF FAILURE BEHAVIOURS OF A DAMAGED REPAIRED STEEL CONCRETE COMPOSITE GIRDER USING CFRP SHEETS

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1. INTRODUCTION

Steel-Concrete composite girders for Highway Bridges became increasingly popular construction method around the world due to the most competitive solution. Large number of steel-concrete composite girder bridges were built in Japan since 1953 because of the need for cost-effective structural forms. However, according to the Ministry of Land, Infrastructure, Transportation and Tourism (MLIT), about 43% bridges in Japan will be more than 50 years older by 2023. These steel-concrete composite girders are required to be upgraded and/or repaired because of the environmental damages, increase in traffic density, lack of proper maintenance and aging [1, 2]. However, the rational design method for repaired steel-concrete composite girders using CFRP sheets for the new construction are enabled because of the revised Design Specification of Highway Bridges 2017 of Japan that has incorporated the limit state design method. It allows the response in the plasticized region be utilized for load-carrying capacity and urges to propose rational design method for CFRP repaired steel-concrete girders for the deficient or damaged bridge components [3]. The past Study showed many experimental tests have been done on the repairing and retrofitting of steel-concrete girders with epoxy bonded or bolted FRP materials. However, no comprehensive investigations are found in the literature that predicts the failure events and complex failure of the CFRP sheets bonded to the tensile flange for repairing a damaged steel-concrete composites girder by FE models.

2. SPECIMEN TESTING

The experimental data was used in this study conducted on two steel-concrete composite girder specimen reference and damaged (denoted as MY1 and R3). The specimen MY1 has no damage on it, on the other hand damage was simulated on the R3 specimen on the mid span of upper face of the lower tensile flange. Damage was represented by reducing a cross-sectional area (1480 mm×4.5 mm). A six-layer CFRP sheets were bonded using epoxy resin adhesive on the tension flange to repair the damage on it. The length of the steel-concrete composite girders was 8.62 m. The dimension of the concrete slab was 200 mm × 500 mm × 8620 mm. The left and right side of the web has a dimension of 2850 mm× 740 mm × 12 mm and at the mid-section of girder the web was 2920 mm× 740 mm × 6 mm. The dimensions of the specimens are shown in Fig. [1]. The specimen MY1 and R3 were similar in dimension.

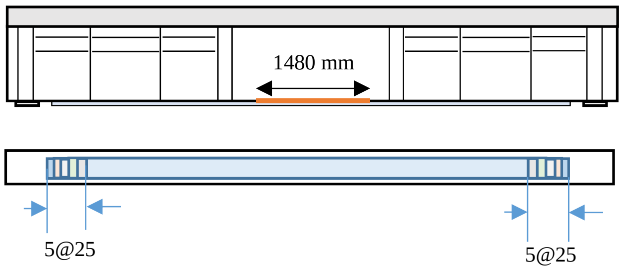

Fig. 1 Damage and CFRP layer attached on tensile flange

3. NUMERICAL MODEL

FE model is developed for MY1 and R3 specimens. The structural steel is modelled using 4-node regular curved shell element Q20SH. The concrete slab, reinforcing bars and CFRP sheets are simulated by using 12-node isoparametric solid element (CHX60), embedded bar elements and 4-node regular curved shell (Q20SH) element respectively. The epoxy resin between two CFRP layers are simulated by 3D interface element (Q20IF). The peeling of CFRP layer is accounted in the FE model by inputting the stress vs. relative displacement of epoxy resin in normal and shear direction. The tensile and compressive behavior of concrete was modelled using JSCE tension softening and Maekawa damage concrete model. The material properties used in the FE analysis are shown in Table 1.

Table 1 Material properties used in FE models

	Parameters	MY1	R3
		MPa	MPa
Concrete	Compressive strength f_c'	37.8	34.9
	Tensile strength f_t	2.99	2.55
	Elastic modulus E_c	29130	30270
Structural steel	Web	424	394
	Upper Flange	405	413
	Lower Flange	392	413
CFRP	Elastic modulus	-	380 GPa
	Tensile strength	-	3365
	Compressive strength	-	2490

4. MODEL VERIFICATION

The finite element models were verified with the experimental results. The load-displacement curves were plotted for the MY1 and R3 specimen. Load-deflection were measured at the middle of the girders and failure modes were compared with measured results. Fig. 2 and Fig. 3 shows the comparison of load-displacement curves between experimental results and the FE analysis results. The maximum load found from experiment and FE analysis for MY1 specimen was 1667 kN

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and 1681 kN. Moreover, for R3 specimen these results were 1683 kN and 1716 kN (Fig. 2 and Fig. 3). The Fig. 2 and Fig.3 shows a good agreement was achieved between experimental and numerical results. The difference in the load -displacement curve for the MY1 and R3 composite girders were very small. Moreover, the trend of the load-displacement curves seen from FE analysis was found similar to the experimental load-displacement curve up to the elastic region. In addition, the elastic and plastic stiffness of the steel-concrete composite girders found from the FE analysis agreed with the experimental results very well. The results show that the difference between the FE analysis and experimental results was in terms of max load was about 1% to 2%, and thus the FE analysis results obtained a high level of accuracy.

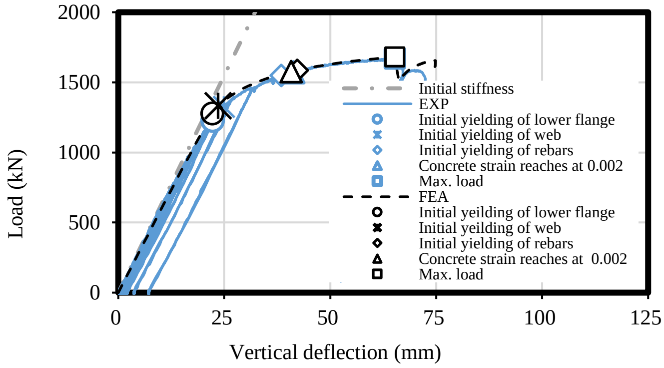

Fig. 2 Comparison of Load-displacement relationship (MY1)

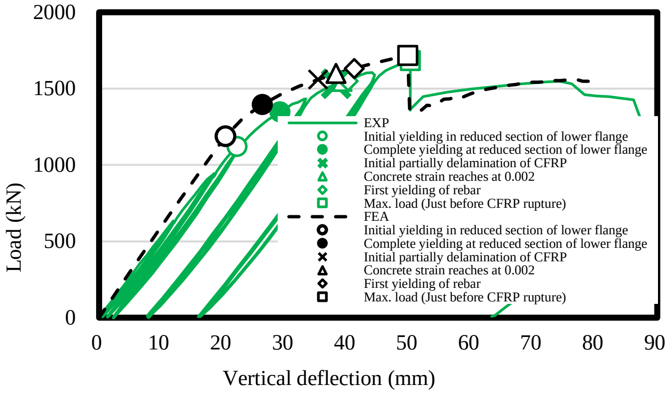

Fig. 3 Comparison of Load-displacement relationship (R3)

5. FAILURE MODES

The failure events were extracted from the FE models and compared with the experimental results. Each of failure events obtained from FE models and experimental results for both specimens are listed in Table 2. The FE analysis shows that the order of the failure events occurred in the same orders as observed in the experiments for both MY1 and R3 specimen. Table 2 shows the failures events taken from the FE analysis and compared with experiments. The difference of moment of occurrence was very small for the failure events of initial yielding of web, concrete strain reaches at 0.002 and for the maximum load. On the other hand, the ratio of FE analysis and experiments was slightly larger 1.04 for initial yielding of lower flange.

Table 2 Comparison of failure events obtained from FE analysis and experiment (MY1 specimen and R3 specimen)

MY1 Spe.	FE analysis		Exp. results		FEA/EXP	R3 Specimen	FE analysis		Exp. results		FEA/EXP
Failure events	Disp. (mm)	Load (kN)	Disp. (mm)	Load (kN)	Load (-)	Failure events	Disp. (mm)	Load (kN)	Disp. (mm)	Load (kN)	Load (-)
Initial yielding of lower flange	22.14	1279	22.2	1227	1.04	Initial yielding in reduced section of lower flange	20.39	1187	22.22	1121	1.06
Initial yielding of web	23.57	1333	24.9	1325	1.00	Complete yielding at reduced section of lower flange	26.32	1395	29.15	1347	1.04
Initial yielding of rebars	42.16	1585	38.4	1548	1.02	Initial partially delamination of CFRP (the edge of reduced cross-section)	35.28	1559	38.22	1530	1.02
Concrete strain reaches at 0.002	40.77	1575	41.3	1571	1.00	First reaching 0.2% strain on slab	38.21	1598	39.18	1547	1.03
Max. load	65.22	1681	65.2	1667	1.00	First yielding of rebar	41.12	1631	40.16	1548	1.05
						Max. load (Just before CFRP rupture)	49.76	1716	50.20	1683	1.02

6. CONCLUSIONS

In this study, numerical analysis results were compared with experimental results for damage repaired girders using CFRP sheets. The numerical analysis showed a good agreement with experimental results. In addition, the predicted failure events matched with the experimental events very well.

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