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Miami Guideway: Testing of Prestressed Twin-Tee Girders

Voie ferroviaire de Miami: essais de poutres précontraintes

Miami Guideway: Versuche an vorgespannten zweifach-T-Trägern

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SUMMARY
Over 2 300 precast, pretensioned, twin-tee guideway girders are being fabricated for Miami's Rapid Transit. Two full size, 85-tonne girders were precracked with 60% overload and then subjected to 6 million cycles simulating 60 years of service life under combined bending, shear and torsion. No crack propagation was observed; this exemplary behaviour is due to the "zero service tension" design and stabilized strands.

RESUME
Plus de 2 300 poutres précontraintes à section en T jumelés ont été fabriquées pour supporter la voie de roulement du "Rapid Transit" de Miami. Deux poutres grandeur nature de 85 tonnes furent préfissurées sous une surcharge de 60% et furent ensuite soumises à 6 millions de cycles simulant 60 années d'utilisation sous flexion, effort tranchant et torsion. Aucune propagation des fissures ne fut observée; ce comportement exemplaire est dû au dimensionnement pour des tractions de service nulles et aux torons faits à partir de fils stabilisés.

ZUSAMMENFASSUNG
Über 2 300 vorgespannte zweifache-T-Träger werden als Fertigteile für die Miami "Rapid Transit" hergestellt. Zwei der 85t schweren Träger wurden mit 60% Vorlast angerissen und anschliessend dynamisch belastet. Die 6 Millionen Lastwechsel entsprechen dabei einer Betriebsdauer von 60 Jahren unter Biege-, Torsions- und Querkraftbelastung. Dabei konnte kein Fortschreiten der Risse festgestellt werden. Dieses Resultat ist auf die Tatsache zurückzuführen, dass einerseits im Gebrauchszustand im Betonquerschnitt keine Zugspannungen auftreten und andererseits die Vorspannlitzen aus stabilisierten Drähten bestehen.

1. INTRODUCTION

Stage I of the Miami Rapid Transit System, scheduled for operation in 1984, is a 36km, heavy rail line, with more than 34km. of elevated dual-track guideway. The precast prestressed concrete industry is very active in Miami and this transit alignment had obvious potential for repetitive span layout. Thus, as expected, a design and cost study established that the most economical aerial guideway superstructure would be a two-ribbed, simply supported prestressed girder, one for each track, precast as a single unit complete with deck slab and end diaphragms, and transported to job site for erection by cranes, one crane in most cases. Optimum spans were set at about 24m and over 2,300 standard girders are required.

As an alternate to the conventional, relatively costly, single-cell box girder with its expensive inside formwork and complex concreting operation, the Consulting Engineers (Kaiser Transit Group) introduced a large precast pretensioned double tee as a guideway girder; this was a 'first' for any U.S. transit system. Besides an impressive saving of over 25% in fabrication cost, mainly due to fixed forms and simple concreting and stripping operations, the open-bottom section was also preferred architecturally as it imparted lightness compared to the massive look of the closed box, supported on relatively short columns.

2. OBJECTIVES OF DEMONSTRATION TESTS

It is prudent to design any guideway girder to have sufficient torsional and flexural strength and ductility to resist abnormal torsional and impact overload caused by the extremely rare case of eccentric loading, initiated by high speed derailment. Under such preset 'design' condition(maximum transverse eccentricity of 92 cm. and arbitrary 100% impact for Miami system), the girder should not crack extensively and remain 'serviceable' thereafter, hopefully without any repairs or with minor repairs. Adequate torsional and bending stiffness and adequate fatigue resistance of girder under repetitive operational (service) dynamic moving loads causing combined bending, shear and torsion are principle criteria of 'serviceability', both for uncracked and cracked girders.

Before the design could be safely endorsed for competitive bidding, test verification was required of all the criteria defined above, and also the following most important design and fabrication aspects:

-Verification of complex elastic (precracking) and postcracking mixed torsion (St. Venant and warping torsion)analyses by special torsion consultant Dr.T. Hsu [1], predicting girder behaviour under operational and derailment loading.
- Verification of design and reinforcement detailing of articulated ends (dapped for one-half overall depth) required for nearly 30% of the girders.
- Verification of an exhaustive analysis computing loss of prestress in Miami environment. Adequate fatigue resistance of the girder was ensured by a design prohibiting any longitudinal flexural tension in the bottom fibre under 'normal' operational load (89% of crush live load and coexisting nosing and centrifugal forces)and permitting a small bottom fibre tensile stress of 1240kN/m^2(180psi) for the very rare case of 'abnormal peak' operational load (100% crush live load maximum nosing and centrifugal forces and peak prevailing winds of 100kmph). Any significant underestimate of loss of prestress will lead to pronounced violation of 'zero service tension' design criterion of fatigue resistance which ensured that cracks due to accidental overload would not reopen under normal loads.
- Verification of stress concentration effects in concrete and steel at and between hold-down points. Fig. 1 is detail of a 'hold-down' located at each of 4 deflection points (0.37L & 0.63L in each stem) of the harped strands; these hold-downs anchor tensioned strands to form and stressing bed.

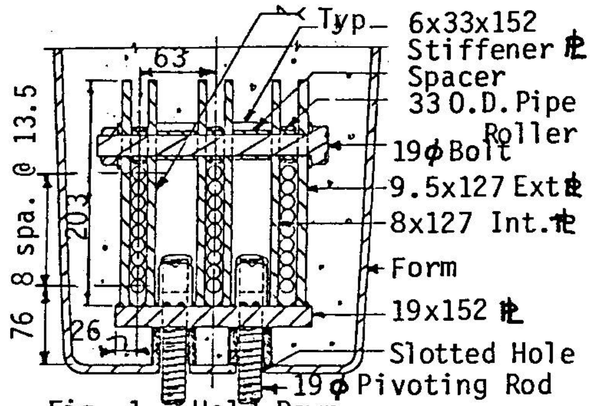

Fig. 1 - Hold-Down

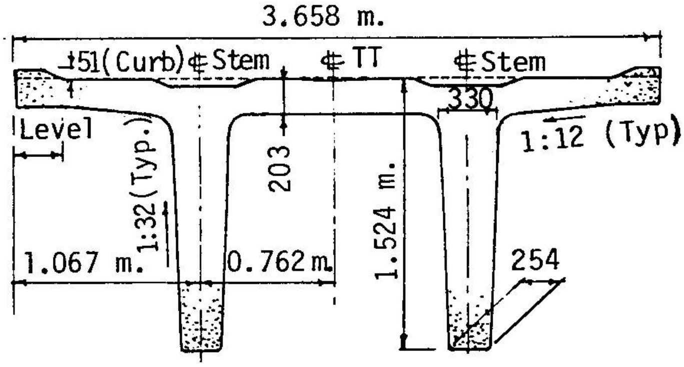

Fig.2-Double Tee Girder-X Section.

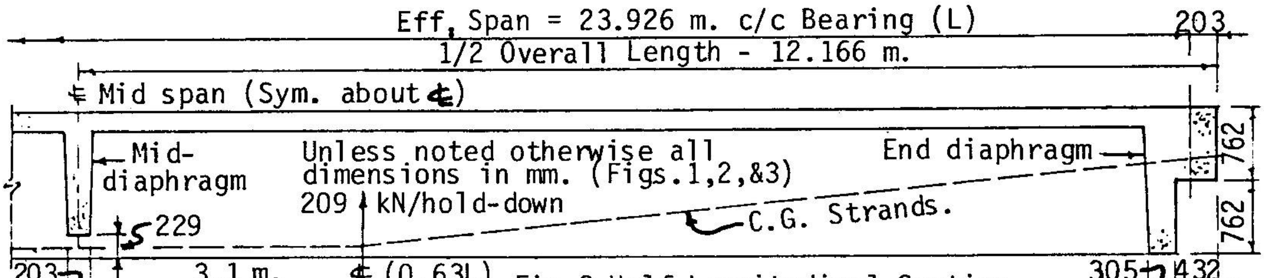

Fig.3-Half Longitudinal Section.

The 54-12.7mm dia. 1,860MN/m^2 stabilized pretensioned strands were divided into 6 vertical columns (3 per stem) of 9 strands each, in contact with each other between hold-down points; the strands are progressively spaced apart to provide a clearance of 85mm between strands at girder ends. Strand contact between hold-downs introduced considerable economy by attaining gain of eccentricity of prestress in the bulbless stem. This hold-down detail was used in the 3 test girders and over 1000 girders fabricated up to October '81. Some engineers felt that the sharp radius of bend at deflection points and the possible rubbing of contact strands over each other may reduce ultimate and fatigue strengths. Yet, the designers were convinced that the 'zero service tension' design and the very small calculated stress fluctuation in the uppermost strands, with the sharpest radius of bend of 17mm and a deflection angle of only 0.12 rad. would guarantee adequate fatigue resistance; note that the second row of strands from bottom had a radius of bend of 110mm (8.7d) for a deflection angle of only 0.01 rad. and a calculated stress fluctuation of only 70MN/m^2(10ksi). The fatigue behaviour of stabilized strands has been established to be significantly superior to stress-relieved strands [2]. The precautions of using stabilized strands and adopting a 'zero service tension' design were strong antidotes used in light of reported fatigue fracture of stress-relieved strands after subjecting precracked State of Louisiana AASHTO type II test beams to only 3 million cycles of repetitive loads creating a design concrete tensile stress of 3100 kN/m^2 (450 psi) [3]; the calculated strand stress fluctuation of 62MN/m^2 (9ksi) matched the calculated value of the Miami girders closely, whereas the calculated minimum strand stress of Louisiana girders (980MN/m^2 = 142ksi) was 10% lower than the Miami girders.

To summarise, the test objectives were to verify all design methods, to confirm fabrication details and to verify the dynamic behaviour of the girders. After extended discussion, it was determined by a blue-ribbon review panel of well-known U.S. experts that only full-size girder testing, as opposed to 1/4 or 1/2 scale models, would yield convincing results for the complex issues involved. As immediate short-term savings of over 10 million dollars were envisaged, and in view of the potential for tapping multiple amounts in future transit systems, a grant of U.S. $350,000 was sanctioned from Research and Design funds of U.S. Department of Transportation for Fabrication and Demonstration Static and Dynamic Tests of two full-size girders and monitoring of a third full-size companion girder for camber and loss of prestress in Miami environment. This paper gives highlights and some details of fatigue testing under repetitive dynamic cyclical loading, tests which are reputed to be the most exhaustive tests in the world on such large full-size girders.

3. TEST SPECTRUM

Special consultant, Dr. J. Fisher of Lehigh University, assisted the Engineers (KTG) in developing suitable test loading spectra of constant stress cycles, which would represent the actual variable stress spectrum of the rapid transit girder. To reduce testing time and costs, it was imperative to reduce the test spectrum to a maximum of 6 million cycles of loading (28 days of continuous loading at 2.5 Hertz average) imposing on the test girder the same cumulative damage as the 16.3 million cycles of actual loading; some basic operational data and the logic of Dr. Fisher's approach [4] is summarised in this chapter.

3.1 Actual Stress Spectrum

The transit system is designed for operation of 2, 4, 6, and 8-car (future option) trains, each 22.85m long, moving on two dual-axle trucks spaced 16.45m apart, with 2/4 car trains used for off-peak and mid-peak hours and 6/8 car trains used for peak hours. Progressive preliminary operating schedules over the projected life of 60 years were analyzed to develop the fatigue design criterion that the girder should sustain loading of 3 million 6-car trains and 1.3 million 2-car trains. Note that for fatigue design purpose, on the conservative side, only 6-car trains were used for peak and mid-peak hours, as these trains cause more cumulative fatigue damage than 8-car trains, for the same total number of cars crossing a girder in any given time. This comparison is based on the premise that a (x+1)- car train with (x) couplers will cause a total of (x) cycles, two 'major' or 'high-loading' cycles (one each on loading and unloading) with maximum fluctuation of strand stress/bending moment/deflection and (x-2) 'minor' or 'low-loading' cycles with only 57% maximum stress fluctuation, neglecting two very minor stress fluctuations; based on Miner's Rule of Cumulative Damages and a conservative low value of n = 3 for the slope of the log S - log N curve, the total cumulative damage of a (x+1)- car train is that due to $[2+(x-2)(0.57)^3]$ major cycles. These general equations are invalid for a 2-car train which creates only one major cycle and no minor cycles; thus, sixteen 6-car trains (96 cars/hour at 3-3/4 mins. headway) would create the cumulative damage of about 41 major cycles, compared to 35 major cyles for twelve 8-car trains (96 cars/hour at 5 mins. headway) handling the same volume of passenger traffic. On this basis, the fatigue design loading of 3 million 6-car trains and 1.3 m. 2-car trains, created 7.3 m. major and 9 m. minor cycles or a major:minor distribution ratio of 55:45. Further the design load distribution specifies 3 subclasses:
- 125,000 six-car trains of abnormal crush load (250 passengers/car), each car weighing 52.4 tonnes (100%), including empty car weight of 35.4 tonnes.
- 1,075,000 six-car and 500,000 two-car trains of 'normal' full load (166 passengers/car) with loaded car weight of 46.7 tonnes (89%) per car.
- 1,800,000 six-car and 800,000 two-car trains of 'mid-peak' load with loaded car weight of 40.7 tonnes (78%) per car.

The 'zero service tension' design criterion applies to 'normal' full loading. The 3 subclasses, when combined with the 57% ratio of major:minor cycle load fluctuation, give an actual spectrum of fatigue load consisting of 6 load ranges (100%, 57%), (89%, 51%) and (78%, 45%), where 100% load or moment fluctuation range is 2827 kN-m (=1.0M)for the major cycle due to abnormal crush load. Using the preceeding equations, the actual spectrum consists of 0.25m. cycles of 1.0M moment range, 2.65m. cycles of 0.89M, 4.4m, cycles of 0.78M, 0.38m. cycles of 0.57M, 3.3m. cycles of 0.51M and 5.4m. cycles of 0.45M, a total of 16.3m cycles.

3.2 Fatigue Testing Load Spectra

The basic concept used for the transformation of the actual stress spectrum into equivalent testing load spectra is Miner's Rule of Cumulative Damages predicting specimen fatigue failure when the cumulative total $\sum f_i N_i$ equals unity, where f_i=number of applications of stress range S_i in the spectrum, and N_i=number of cycles of this same range S_i which alone will cause fatigue failure. When the well-known linear logS-logN relationship (slope=n)for various stress ranges S_i is superposed on Miner's Rule, it can be easily established that any two loading spectra having equal cumulative indices of$(\sum f_i S_i^n)$ will cause the same degree of fatigue damage in the specimen. Note that although values of n=3.56 and 4.53 have been derived from previous tests, a lower value of n=3 was selected, as the consequently steeper slope gives a larger number of test cycles of amplified loading. Further, bending moment fluctuation range ΔM was used in place of stress-range S_i so that the Index of Cumulative Damages for the actual and test loading spectra was taken as $\sum f_i (\Delta M)^3$ which totalled $(114.5x10)(MN\text{-}m)^3$ for the actual variable stress spectrum defined in 3.1 above. The following 3 equivalent test loading spectra were determined to generate the same index.

Test Spectrum	Total No.of Cycles	No.of Cycles-Range M	No.of Cycles-Range 0.89M
Alternate A	5.05 Million	5.05 Million	None
Alternate B	6.00 Million	3.20 Million	2.80 Million
Alternate C	7.07 Million	0.25 Million	6.82 Million

A cycle of range M=2.827MN-m corresponds to the moment fluctuation range of the major cycle caused by 100% abnormal crush load, whereas the (0.89M) fluctuation range cycle corresponds to the major cycle of 'normal' full loading. Note that the substitution of(S_i) by(ΔM) in the equation for Index of Cumulative Damages is permitted because of the linear relationship between fibre stress and bending moment over virtually the entire loading range concerned. Based on uncracked section properties, the maximum bottom fibre tensile stress under 100% abnormal crush load is only about 800kN/m^2(120psi), including the longitudinal tension in the bottom of one stem of the girder due to warping torsion bimoment created by coexisting additive transverse horizontal forces (nosing, centrifugal force and a sustained wind of 20 kmph). Thus, even if the section has been previously cracked by an extreme overload, the reopening of the crack under 'abnormal crush load' will be very small and the penetration of the crack will be very shallow. In this way, it was reasoned that treating the behaviour as linear throughout would introduce only negligible errors, on the conservative side, in determining alternates A and B; of course alternate C, which retained the 0.25m. cycles of range M, would generate more exact equivalence between actual and test spectra.

4.0 FABRICATION OF TEST GIRDERS

Figs. 2 and 3 give the cross-section and one-half longitudinal section of a prototype standard girder designed for tangent track with webs stiffened by a midspan diaphragm and dapped end diaphragms, all precast as a single unit. Three full-size test double-tees, 24.34m long by 3.66m wide by 1.52m deep were made to dimensions identical to the prototype with the exception that the complex contoured top surface, with edge curbs and recesses, was replaced by a level surface (shown dotted in Fig.2) to facilitate installation of test loading equipment and to reduce cost of fabrication; this minor change made a nominal difference in the critical flexural stress history of the bottom fibre. The test girders were fabricated by Stresscon, Miami, using a special, relatively inexpensive, self-stressing concrete and steel form. Although Miami did not have a suitable testing laboratory, the test girders were made in Miami rather than at a casting yard near five potential laboratories located in California, Illinois, Pennsylvania and Texas, due to the following reasons:

- The 2,300 prototype girders would most likely be made in Miami using local South Florida oolitic limestone coarse and fine aggregates for the concrete.

-Fabrication of test girders outside South Florida necessitated shipping of local aggregate to a casting yard near the potential testing lab, to match relatively low moduli of elasticity and rupture and high specific creep strain of oolitic concrete. Also, one test girder (No. 1) would have to be shipped to Miami to simulate actual storage conditions of temperature and humidity.
-Inspection for total conformance of details and specifications was essential.
-Total costs were the least for girders made in Miami and shipped to test lab.

The strands of test girder No. 1 were initially tensioned to the full design value of 130 kN per strand between hold-downs, as per prototype design, computed to give corresponding prestress of 101 kN after all losses. Strands of test girders No. 2 and 3 were initially tensioned to lower values of 123 kN and 127 kN respectively, computed so that prestress available at the scheduled time of commencement of fatigue testing, is 101 kN per strand.

The following special test equipment was incorporated in the test girders:
-Load cells mounted on the strands between anchorage and stressing abutment monitored loss of tension between initial tensioning and start of transfer of prestress. Thereafter, the actual loss of prestress was computed from change in strain recorded by Whittemore and Demec demountable mechanical strain gauges, applied to surface discs preglued on gauge lengths, located on both webs at the c.g. of strands and 3'-6" either side of midspan. The observed readings were compensated for changes in temperatures of the concrete surface and the gauges.
-Twentyfour SR-4 electrical resistance strain gauges were epoxy glued to the first, second, fourth, sixth and top row of the inner column of tensioned strands of both webs of test girder No 2 only; these strain gauges were also located 3'-6" from midspan. Five of these gauges were continuously monitored during the fatigue test. To force cracks at gauge lengths, a crack former steel plate was placed at each location between web soffit and underside of strands.
-Thirtyone SR-4 gauges were epoxied on stirrups in stems between 0.05L and 0.20L and in an end diaphragm (0.0L) of test girder No 3 only.

5.0 DYNAMIC TESTS

Transportation of girders from Miami to California or Pennsylvania presented almost insurmountable problems. After extended evaluation of all parameters, it was decided to test Girders Nos. 2 & 3 at Portland Cement Association, Skokie, Illinois; these girders were transported by rail from Miami to Skokie siding.

5.1 Test Loading and Monitoring Equipment

Two MTS 490 kN (110 kips) actuators, each representing loading from a dual-axle truck located nearest the coupler of a pair of married cars, were used for repetitive loading. The piston of each actuator was clamped to the girder as shown in Fig 4; note that swivel heads were used at top and bottom ends of the actuator and connected to steel cross heads, with the top cross head clamped to the girders by tie-rods and the bottom cross head anchored to laboratory floor. This efficient arrangement enabled use of the available actuators and reduced the cost of the hydraulic system. For girder No. 2, with actuators located 3.20m on either side of midspan, each tie-rod was prestressed to 266 kN and thus will not affect the results of the symmetrically loaded girder with 'coupler at midspan'. However, as the coupler is located at 0.21 L for girder No 3 providing unsymmetrical load, the rods were not prestressed, but a different clamping system devised to avoid external prestressing forces on the girder webs. The girder is under flexure and vertical shear when the centre-lines of the actuator and the girder are aligned; transverse 'eccentricity' of the girder makes P1 and P2 unequal and then the girder is also subjected to torsion.

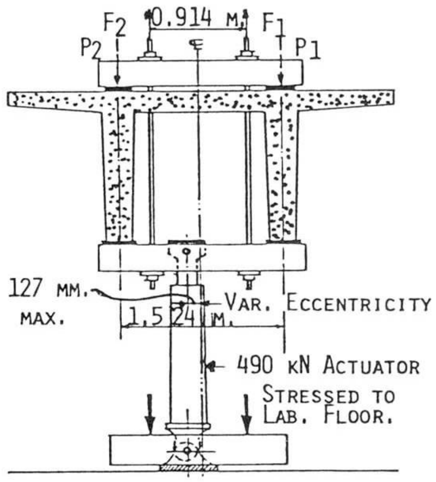

Fig.4-Repetitive Loading Setup

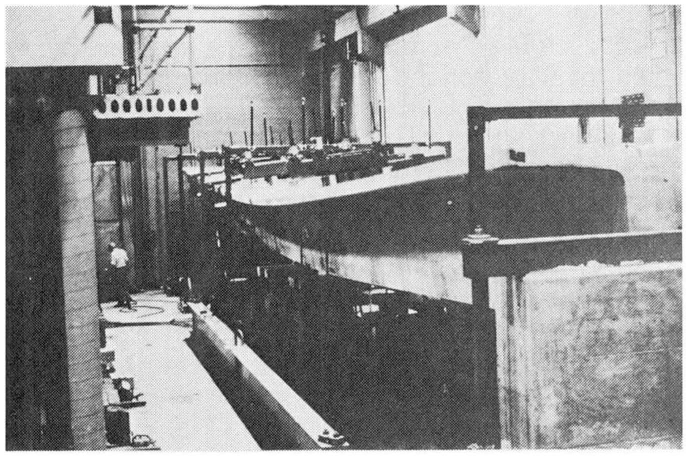

Fig. 5 - Girder No. 2 at Ultimate Load.

The computed first mode natural frequencies of the girder, without the mass of the design superimposed load which was applied as external test loading, are 4.5 cycles per second in flexure(vertical impulse) and 5.9 cps in torsion(horizontal impulse) ignoring, on the safe side, warping torsion stiffness. The 490 lpm (120 U.S. gpm) hydraulic system was estimated to move enough oil to pulsate girder No 2 at 2.4 to 2.7 Hertz under maximum loading, with a calculated dynamic deflection range of 18 mm, and to pulsate girder No. 3 at 3.0 to 4.0 Hz with a smaller deflection range; so, no resonance problems were expected. However, during testing of girder No 3 at 3.3 Hz, there was some minor resonance with the laboratory building and this particular frequency was avoided thereafter.

Precracking of girder No 2, preceeding flexural fatigue testing, was done with loads applied at 0.37 L and 0.63 L through a system of cross heads and the rods, without using the actuators. Precracking of girder No 3, which required larger loads applied at 0.08 L and 0.34 L to produce cracking, was done by using the actuators together with an auxiliary ram applying loads through cross-heads.

Monitoring of fatigue tests was done as follows:
-Cracks were marked with black felt tip pens, photographs taken and widths measured by illuminated hand microscopes.
-Demountable mechanical strain gauges were used to compute available prestress.
-A linear variable differential transformer (LVDT) was attached to brackets glued to the concrete surface on either side of the worst crack of girder No. 2.
-A six-channel Sanborn strain recorder monitored SR-4 gauges and LVDT.
-Minimum and maximum loads, corresponding deflections sensed by 50mm travel potentiometers, and cyclic frequency were registered on MTS digital indicators.
-During precracking, data from load cells, rotation gauges, potentiometers and SR-4 gauges was recorded by a Vidar digital data acquisition system and fed into a mini-computer for conversion into engineering units.

5.2 Test Loading of Girder No. 2

This is a test load creating maximum flexure with corresponding shear and torsion. The girder was precracked under a total symmetrical load of 1192 kN, applied equally to both webs at 0.37L and 0.63L, which equals superimposed dead load plus 1.6 times abnormal crush load including impact. This overload cycle, which caused several cracks up to 0.5m long and 0.15 mm wide within the constant moment zone, was repeated 5 times to stabilize strand movements. Thereafter, this girder was subjected to 6 million cycles of loading as per test sprectrum alternate B; transverse eccentricities imposed torsion as required and direction of torsion was reversed after 50% of the cycles. There were 6 loading stages, with maximum moment range cycle 1.0 M used for the commencement and at the end of the test. The large mass of the girder necessitated a dynamic correction accounted for by controlling the loads such that the deflections produced by cyclic loading matched measured minimum and maximum static deflections of both webs. Control static deflections were remeasured several times.

5.3 Test Loading of Girder No. 3

This load created maximum combined shear stress (due to vertical shear and St. Venant torsion) with coexisting flexure. Precracking loads of 436 kN were applied through each of the two actuators kept at 0.08L and 0.34L, augmented by an auxiliary load of 427 kN applied at 0.375L. This process produced several flexural cracks between 0.28L and 0.44L, the largest being about 0.6 m long and 0.2 mm wide. Thereafter, the auxiliary load was removed, actuator positioned to required transverse eccentricities and the girder subjected to 5.05 million cycles of 100% crush loading with coupler located at 0.21L (Test Spectrum A).

6.0 FATIGUE TEST RESULTS

6.1 Test Girder No. 2

Results of fatigue test on girder No 2 are summarized below:
-Static deflection tests conducted periodically during fatigue testing showed no apparent deterioration of flexural stiffness.
-The maximum crack width of 0.035 mm under maximum moment range increased to only 0.05 mm after 6 million cycles of loading.
-The maximum measured strand stress increase was about 50 MN/m^2 (8ksi) matching calculated values closely.
-After fatigue testing, the girder was subjected to a static load equal to 2.25 design derailment load or 1.6 times the required ultimate strength. Figure 5 is a photograph of the twisted girder at incipient failure.

6.2 Test Girder No. 3

Results of fatigue test on Girder No 3 are summarized below:
-There was no apparent deterioration of torsional or flexural stiffness.
-Flexural cracks did not propogate during the fatigue test.
-Measured stirrup strains showed that concrete did not crack in combined shear.
-Ultimate strength was 2.5 times derailment load with coupler at 0.21L.

7.0 CONCLUSION

Flexural fatigue resistance was fully established by Girder No 2. Test data provided by Girder No 3 clearly established torsional fatigue resistance.

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