Background of American design procedure for fatigue of concrete

Autor(en): **Gene Corley, W. / Helgason, Thorsteinn / Hanson, John M.**

Objekttyp: **Article**

Zeitschrift: **IABSE reports = Rapports AIPC = IVBH Berichte**

Band (Jahr): **37 (1982)**

PDF erstellt am: **15.08.2024**

Persistenter Link: https://doi.org/10.5169/seals-28892

Background of American Design Procedure for Fatigue of Concrete

Principe de la méthode américaine de dimensionnement à la fatigue du béton

Über das amerikanische Ermüdungsbemessungsverfahren für Betontragwerke

W. GENE CORLEY
Director
Portland Cement Assoc.
Skokie, IL, USA

JOHN M. HANSON
President
Wiss, Janney, Elstner & Assoc.
Northbrook, IL, USA

THORSTEINN HELGASON
Associate Professor
University of Iceland
Reykjavik, Iceland

SUMMARY
The procedure currently used in North America for considering fatigue of reinforcement in concrete highway bridges was adopted in 1975. The procedure was based on an extensive experimental investigation to determine the fatigue strength of U.S. manufactured, hot rolled, deformed, reinforcing bars. This paper summarizes results of the test program and presents the statistically developed design procedure. Additionally, an example of the use of this provision in American practice is provided.

RESUME
La méthode couramment utilisée en Amérique du Nord pour prendre en considération la fatigue des aciers d'armature dans les ponts en béton fut adoptée en 1975. Cette méthode est basée sur une vaste étude expérimentale, entreprise en vue de déterminer la résistance à la fatigue des barres d'armature fabriquées aux USA, en acier laminé à chaud et déformé. Cet article résume les résultats expérimentaux et présente la méthode de dimensionnement développée sur une base statistique. On présente en outre un exemple pratique d'application de cette méthode.

ZUSAMMENFASSUNG
Das zur Zeit in Nordamerika angewandte Verfahren zur Berücksichtigung der Ermüdung der Stahleinlagen in Betonbrücken kam erst 1975 in Gebrauch. Das Verfahren wurde aufgrund einer umfangreichen experimentellen Untersuchung zur Ermittlung der Dauerfestigkeit der in den USA hergestellten Rippenstähle entwickelt. Die Ergebnisse der Versuche und das statistisch entwickelte Verfahren werden im Aufsatz beschrieben. Im weiteren wird ein Beispiel zur Anwendung des Verfahrens für amerikanische Verhältnisse angegeben.

1. INTRODUCTION

Early bridge design specifications did not need to consider fatigue because 125 to 140 MPa allowable design stresses were too low to present a danger of fatigue fracture. To date, no fatigue damage of a concrete bridge in regular service has been identified. However, Grade 40 bars were placed in two reinforced concrete test bridges [1] in a Road Test conducted by the American Association of State Highway Officials (AASHO) in the late 1950's. Some of these bars fractured in fatigue after repeated application of very heavy loads to the bridges following completion of the planned field tests.

More recently, high yield stress reinforcing bars have come into usage, load factor design methods are permitted, and heavier trucks are allowed on North American highways. Together, these factors result in repeated stresses that approach those known to cause fatigue fracture in reinforcing bars.

In 1974, American Concrete Institute (ACI) Committee 215 published a state-of-the-art report on fatigue of plain and reinforced concrete [2]. Based on data available at that time, the Committee recommended that the stress range for straight deformed reinforcing bars be limited to 145 MPa. This limit was adopted by AASHTO in the 1974 Interim Specifications [3].

In 1976, results of an extensive investigation [4] to determine the fatigue strength of U.S. manufactured hot rolled deformed reinforcing bars were published. This investigation was carried out by the Portland Cement Association (PCA) and sponsored in part by the National Cooperative Highway Reseach Program (NCHRP). The work included a review of the literature, 353 tests on bars embedded as a single reinforcing element within a concrete beam, and a statistical analysis of the resulting data.

2. TEST VARIABLES AND PROCEDURE

Bars shown in Fig. 1, from five U.S. manufacturers, were tested. One of the manufacturers was represented by nominal 16, 25, and 33 mm bars having guaranteed yield stresses of 276, 414, and 517 MPa and by nominally 19 and 32 mm bars having a guaranteed yield stress of 414 MPa. The other manufacturers

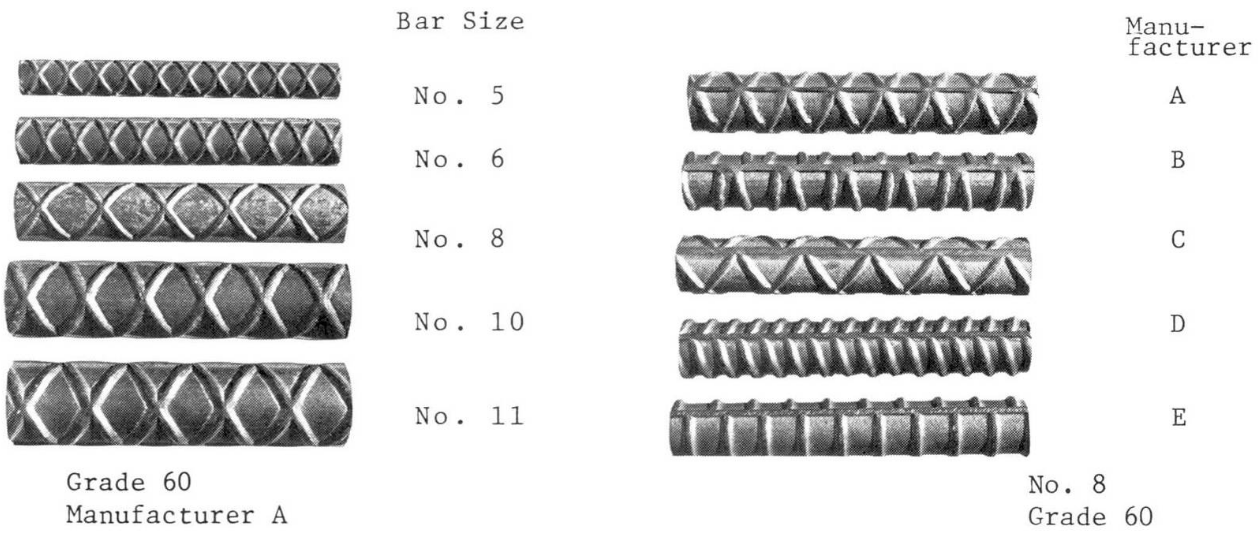
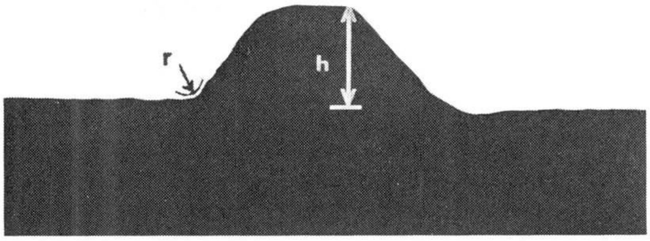


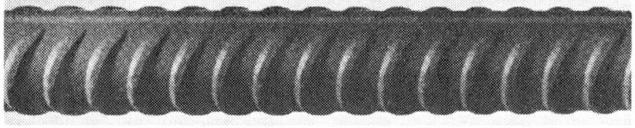
Fig. 1 Reinforcing Bars Used in Test Program

were represented only by nominally 25 mm bars having a guaranteed yield stress of 414 MPa. Each manufacturer's bars had a distinctive rib pattern.

At the time that the investigation was carried out, hot-rolled deformed bars were made by approximately 50 manufacturers in the U.S. These bars have a wide variety of deformation patterns. Typically, the patterns include two longitudinal ribs and transverse lugs either perpendicular to the ribs or inclined at an angle of not less than 45 degrees with the longitudinal axis of the bar. Bars that were tested represented a range of transverse lug geometries. Lug geometry was assessed by means of the ratio of base radius to rib height, r/h, defined in Fig. 2. This ratio ranged from 0.17 to 0.39 for the five manufacturer's bars.



(a) Lug Profile



(b) Lugs and Rib

Fig. 2 Base Radius r and Lug Height h

Bars were tested by embedding them in a concrete beam and subjecting the beam to repeated loads. Each test beam was rectangular or T-shaped in cross section and had a nominal effective depth of 150, 250, or 450 mm. In Fig. 3, one of the beam tests is shown.

Loads were applied to the test beams using either one or two 100 kN capacity Amsler rams, depending on beam size. Loads varied sinusoidally and were applied at fixed nominal rates of either 4.2 or 8.3 Hz. In each case, applied loading produced a constant moment region in the central 1/3 of beam span. Each test beam was simply supported on rollers.

Tests were carried out in two series. In each series, order of testing and selection of bars was randomized to obtain a statistically valid test program.

Stress range in bars of Series 1 tests was varied to obtain an S-N curve in the finite life region for each test condition. Minimum stress levels were nominally 41 MPa compression, 41 MPa tension, and 124 MPa tension in finite-life tests of Series 1. When minimum stress in a test bar was compression, external post-tensioning was applied to the test beam. The post-tensioning system consisted of a pair of steel rods held at the level of the beam reinforcement and passed through steel springs butting against one end of the test beam. Prestress force was measured by load cells.

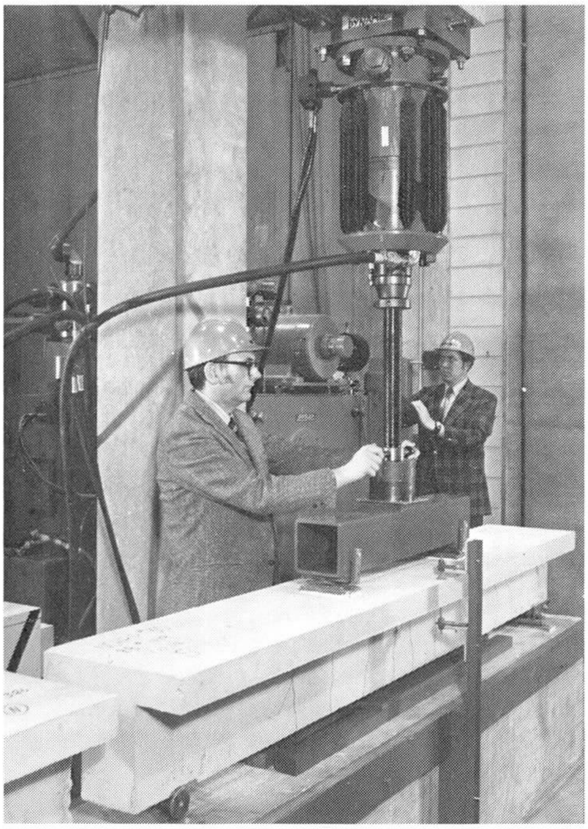


Fig. 3 Test Setup

In Series 2, stress range in bars was varied in increasing or decreasing steps of 7 MPa to obtain a series of staircase results around a fatigue limit of 5 million cycles. Stress range and rib geometry were the only variables in the staircase test series. All tests of Series 2 were conducted on nominally 25 mm bars, each embedded at an effective depth of about 250 mm and subjected to a nominal minimum stress of 41 MPa tension. In the staircase tests, stress range applied to a specific manufacturer's test bar depended in each case on results obtained in the immediately preceeding test on that manufacturer's bars. Thus, a runout at 5 million cycles resulted in a nominal 7 MPa increase in stress range for the succeeding test. Conversely, a fatigue fracture in a test bar resulted in a nominal 7 MPa decrease in stress range for the succeeding test.

3. TEST RESULTS

3.1 Finite-Life Tests

Stress range was found to be the predominant factor affecting fatigue life of each reinforcing bar. Statistical analysis of the test data showed that next to stress range, minimum stress level was the variable of greatest significance. Increasing minimum stress from compression through tension caused a statistically significant reduction in fatigue life. Effects of these and other test variables are presented in detail in Ref. 4.

Considering the effect of stress range alone, the relationship between the logarithm of fatigue life and stress range was found to be:

$$\log N = 6.9690 - 0.0383 f_r \tag{1}$$

where: N = fatigue life

f_r = stress range at centroid of reinforcing bar during stress cycle

This relationship explained 76.8% of the variation in test data. The standard deviation for the regression was 0.16557.

During the tests, 33 mm bars having a yield stress of 414 MPa fractured in fatigue after 1,250,000 cycles when subjected to a stress range of 147 MPa and a minimum stress of 121 MPa. This is the lowest stress range at which a fatigue fracture has been obtained in a straight U.S. manufactured bar. Fatigue fractures have been obtained at lower stress ranges in bent or welded bars.

Tests conducted at low stress ranges indicated that there is a limiting stress range, the fatigue limit, above which a bar is certain to fracture in fatigue, and below which a long fatigue life is possible. Subsequent tests [5] have confirmed that below the fatigue limit, a reinforcing bar may be able to sustain a virtually unlimited number of cycles of loading without fracture.

3.2 Staircase Tests

For the five manufacturer's bars tested, mean fatigue limit at 5 million cycles was found to range from 159 to 197 MPa. This variation had a strong correlation to the rib geometry factor, r/h. Assuming a normal distribution of data around each mean fatigue limit, upper and lower tolerance limits were established, with 95% probability that 95% of all possible test results on a particular manufacturer's bars would fall within the limits. For the five manufacturer's bars, the lower tolerance limit ranged from 136 to 184 MPa.

A linear regression analysis performed on the results of the staircase analysis, using fatigue limit, f_f, as the dependent variable, resulted in the following relationship:

$$f_f = 7.88 + 52.85(r/h) \qquad (2)$$

However, this expression may place an undue emphasis on the effect of bar geometry since the effects of other potential influencing factors such as minimum stress level could not be considered.

4. FATIGUE DESIGN PROVISION

The current design provisions [6] of the American Association of State Highway and Transportation Officials (AASHTO) and the ACI-ASCE Committee on Bridge Design [7] require that stresses at service loads in reinforced concrete bridges shall be limited to the following:

4.1 Concrete maximum compressive stress shall not exceed $0.5f'_c$ at sections where stress reversals occur caused by live load plus impact at service load. This stress limit shall not apply to concrete deck slabs.

4.2 Reinforcement range between a maximum tension stress and minimum stress in straight bars caused by live load plus impact at service load shall not exceed:

$$f_f = 145 - 0.33f_{min} + 55(r/h)$$

where: f_f = stress range, MPa

f_{min} = algebraic minimum stress level, tension positive, compression negative, MPa

r/h = ratio of base radius to height of rolled on transverse deformation; when actual value is not known, use 0.3

Bends in primary reinforcement shall be avoided in regions of high stress range.

An example of application of these design provisions is given in the Appendix. Provisions apply only to straight hot-rolled bars with no welds and with no stress raisers (including manufacturers marks) more severe than deformations meeting the requirements of American Society for Testing and Materials (ASTM) Designation: A615.

5. DESIGN EXAMPLE

Application of the fatigue design provision is illustrated in the following partial design calculations for the main reinforcement in a bridge superstructure. For simplicity, a slab bridge was selected.

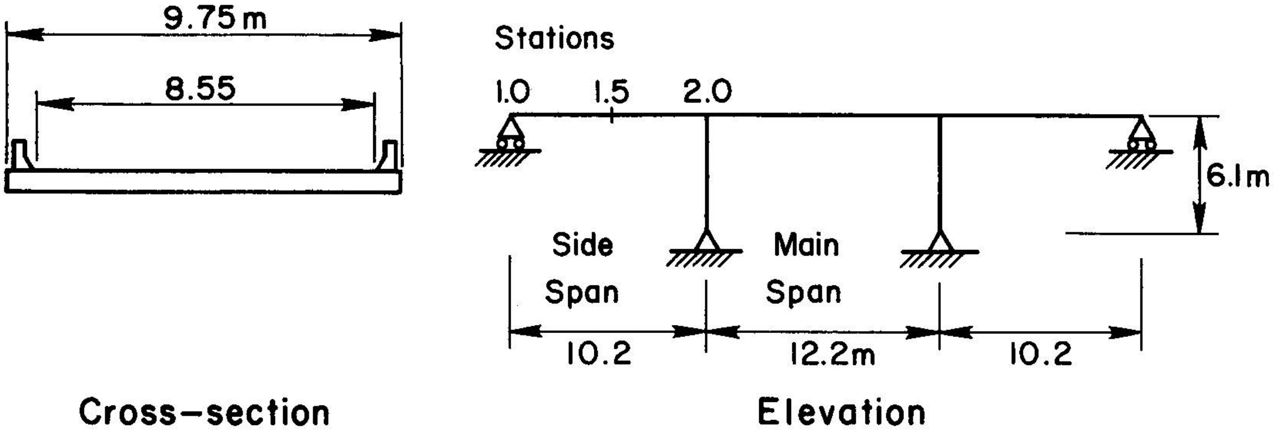


Fig. 4. Example Bridge

This 2 lane highway bridge is designed for HS 20-44 loading [6]. Material properties used in the design are f'_c = 20 MPa for concrete and f_y = 413 MPa for deformed reinforcing bars, assumed to have a lug base radius to lug height ratio, r/h, of 0.3. A 450 mm slab thickness was selected on the basis of maximum reinforcement and deflection criteria. From this and the HS 20-44 loading with an appropriate impact factor, dead load and live load effects at various stations along the span were calculated.

Concrete cover was 50 mm over negative moment reinforcement to account for severe exposure and 25 mm over positive moment reinforcement to account for moderate exposure. Reinforcement at maximum moment locations was selected on the basis of:

$$a = \frac{A_s f_y}{0.85 f b} \qquad M_u = \phi A_s f_y (d - 0.5a)$$

where ϕ is a capacity modification factor equal to 0.90 for flexure. At the centerline of the main span, station 2.50, the initial bar selection was:

$$25 \text{ c/c } 195 = 2518 \text{ mm}^2/\text{m}$$

$$a = \frac{2518 \times 413}{0.85 \times 20 \times 1000} = 61.2 \text{ mm}$$

$$M_u = 0.90 \times 2518 \times 413 \times (412 - 0.5 \times 61.2) = 357.0 \text{ kNm/m}$$

This selection was then checked against crack control requirements and found to be satisfactory. Next, stress range at service loads was calculated:

$$f_{sr} = f_{smax} - f_{smin} = \frac{M_{wmax}}{A_{sjd}} - \frac{M_{wmin}}{A_{sjd}}$$

$$= \frac{190.4 \times 10^6}{2518 \times 0.902 \times 412} \quad \frac{29.7 \times 10^6}{2518 \times 0.902 \times 412} = 203.5 - 31.7 = 171.8 \text{ MPa}$$

and checked agaist the allowable stress range:

$$f_f = 145 - 0.33 f_{smin} + 55 \text{ r/h}$$

$$= 145 - 0.33 \times 31.7 + 55 \times 0.3 = 151.0 \text{ MPa} < 171.8$$

The required reinforcement is:

$$A_s = \frac{2518 \times 171.8}{151.0} = 2865 \text{ mm}^2/\text{m}$$

an increase of 14% from the requirement for strength alone:

Similarly, the required reinforcement in the side span was:

$$22 \text{ c/c } 130 = 2923 \text{ mm}^2/\text{m}$$

providing a moment capacity of 411.2 kNm/m. There, the allowable stress range was exceeded at stations 1.3 to 1.6, for the original bar selection. Due to lack of symmetry in the side span moment diagrams, the critical fatigue location cannot be presumed to coincide with the critical strength location.

Selection of the reinforcement may be summarized in tabular form. Appropriate extensions for development must be provided. Minimum reinforcement and bar spacing considerations permit the main span positive moment reinforcement to be reduced by thirds. Every third bar may be terminated for strength at station 2.31. The cut bars are adequately developed from that point and crack control criteria are satisfied. Service load moment reversal takes place at station 2.31 with:

$$M_{wmin} = 11.4 \text{ kNm/m} \qquad M_{cr} = 93.5 \text{ kNm/m}$$

Therefore, the stress state can be determined as:

$$f_{smax} = \frac{M_{wmax}}{A_{sjd}} = \frac{139.8 \times 10^6}{1949 \times 0.912 \times 414} = 190.0 \text{ MPa}$$

$$f_{smin} = \frac{nM_{wmin}(d - h/2)}{I_g} = \frac{10 \times (-11.4) \times (414 - 225)}{7594} = 2.8 \text{ MPa}$$

$$f_{sr} = f_{smax} - f_{smin} = 190.0 - (-2.8) = 192.8 \text{ MPa}$$

$$f_f = 145 - 0.33 \times (-2.8) + 55 \times 0.3 = 162.4 \text{ MPa} \quad 192.8$$

The location where the bar may safely be cut for fatigue can be determined by trial and error or estimated from:

$$\frac{M_{wmax}}{A_{sjd}} \; 1 - 2.4 \; n \; \frac{M_{wmin}}{M_{wmax}} = 145 + 55 \; r/h$$

which is derived by setting $f_{sr} = f_f$ and making liberal use of the approximation $d = 0.9h$. Using momemt from station 2.31:

$$\frac{M_{wmax}}{1949 \times 0.912 \times 414} \; 1 - \frac{2.4 \times 10 \times 4.71 \times (-11.4)}{1000 \times 139.8} = 145 + 55 \times 0.3$$

$$M_{wmax} = 117.8 \text{ kNm/m}$$

which is found, by linear interpolation, to occur at station 2.277. A check shows that at station 2.275, f_{sr} exceeds f_f by only 0.5%, which is satisfactory. Other cut offs may be determined similarly.

In summary, checks for fatigue must be made at every stage of the design process. Greatest economy in design effort is obtained by bringing fatigue control directly into the reinforcement selection process, as illustrated here. Fatigue requirements may result in a need to increase reinforcement area beyond that required for strength and/or to extend bar cutoff locations.

6. CONCLUDING REMARKS

The design provision for fatigue in the current AASHTO specifications was initially adopted in 1974. This provision was based on an extensive investigation summarized in this paper. In this provision, the limiting stress range in reinforcing bars depends on the minimum stress level and the ratio of base radius to height of the transverse lugs.

7. REFERENCES

1. American Association of State Highway Officials: The AASHO Road Test: Report No. 4 - Bridge Research, Special Report No. 61D, Publication No. 953, Highway Research Board, Washington, D.C., National Academy of Sciences, National Research Council, 1962.

2. ACI Committee 215: Considerations for Design of Concrete Structures Subjected to Fatigue Loading, American Concrete Institute Journal, Proceedings, Vol. 71, No. 3, March 1974, pp. 97-121.

3. American Association of State Highway and Transportation Officials: Interim Specifications: Bridges, 1974, Washington, D.C., 1974.

4. Helgason, Th., et al., Fatigue Strength of High-Yield Reinforcing Bars, NCHRP Report No. 164, Transportation Research Board, Washington, D.C., 1976. (Also PCA RD045.01D).

5. Anon., Concrete International, American Concrete Institute, Vol. 2, No. 12, Dec. 1980, p. 11.

6. American Association of State Highway and Transportation Officials: Standard Specifications for Highway Bridges, Twelfth Edition, Washington, D.C., 1977.

7. "Analysis and Design of Reinforced Concrete Bridge Structures," ACI Committee 443, American Concrete Institute, Detroit, Michigan, March 1977.