IX CONGRESSO BRASILEIRO DE PONTES E ESTRUTURAS
18 a 20 de maio, 2016 - Everest Rio Hotel
IXCBPE
Rio de Janeiro | Cidade Olímpica

Damage assessment for the structural rehabilitation of a trussed bridge on Ferrovia do Aço Railway.

Fernando Rodrigues[1], Pedro Almeida[2], Rodolfo Montoya[3], Marcelo Botelli[4], João Junqueira[5], Felipe Masini[6]

[1,2]Especialistas / LSE /Engenharia / fernando.rodrigues@lsetech.com.br
[3] Consultor / Montoya Alves Engenharia / Engenharia / montoyaeng@gmail.com
[4,5]Especialistas Ferroviário / MRS /Engenharia / marcelo.botelli@mrs.com.br, joão.junqueira@mrs.com.br
[6]Gerente de Engenharia / MRS /Engenharia / felipe.masini@mrs.com.br

Abstract

During visual inspection of Fumaça Waterfall bridge, systematic cracks were identified on I section beams of the deck's support structure. Because damage had such a broad reach an emergency procedure was adopted, including reduction of train speed and study of mechanisms of crack formation to provide an emergency action and to assist a reinforcement design. The mechanisms of crack formation were investigated by means of monitoring and simulations on finite element models. Structural monitoring during train's passages under controlled speeds was accomplished and data collected were suitable to stablish an appropriate train speed limit during the emergency actions. To increase remaining-life of structure to more than 50 years, fatigue study was done using a hybrid method based on F.E.M. values and experimental values. Finally, a reinforcement design based on a redistribution of forces was proposed. This reinforcement restores structure durability to more than 50 years.

Keywords

Railway Bridge; Monitoring Stress; Fatigue, F.E.M.

Introduction

Ferrovia do Aço Railway was constructed in the 1980s as a connection between the most economically prominent states of Brazil, to enable transportation of iron ore from reserves of Minas Gerais to the ports of the São Paulo and Rio de Janeiro's coastlines. Since then, this railway has transported a growing amount of iron ore. Fumaça Waterfall bridge is the only steel bridge on the railway. Located between two tunnels this bridge is one of the strategic bridges for the railway considering the difficulty of access to accomplish any repair on its structure. This way MRS, the concessionaire of the railway, has demanded a structural assessment of the bridge.

Description of the bridge

Fumaça Waterfall bridge is a 70m span truss bridge. Two planar steel Pratt trusses, 12m in height, separated from each other by a distance of 6.2m, assembles the main structure. Trusses are composed by I sections connected with black bolts. (Figure 1) The deck is of open grillage that supports reinforced concrete boxes where railway tracks are rested. Two longitudinal joists (I section) and ten transverse joists (I section) compose the grillage, i.e. the structure that supports the concrete boxes. Connection between longitudinal and transversal joists is done with angle sections, welded to the web of longitudinal joists and bolted to the web of transversal joists. Connection between longitudinal and transversal joists to concrete deck is done with shear connectors (studs).

Figure 1 - Truss bridge over the Fumaça Waterfall

Bridge structural assessment - Visual inspection

During visual inspection of Fumaça Waterfall bridge, systematic cracks were identified on longitudinal and transversal joists as well as on connectors of the deck's support structure. An advanced state of damaged was identified on the transversal joist of the extremity, including rupture on welds between top flange and web on the bearing region and around web stiffeners, as presented on Figure 2. Typical cracks identified on the web of longitudinal joists in the regions of connection with transversal joists are presented on Figure 3.

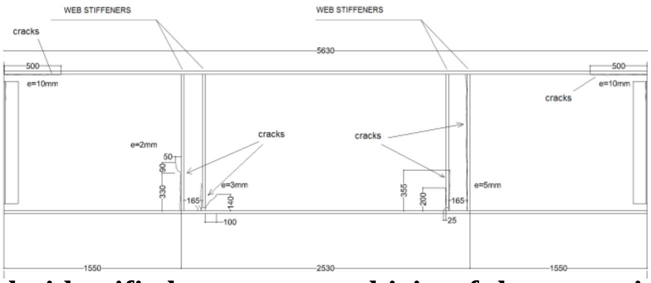


Figure 2 - Cracks identified on transversal joist of the extremity of the bridge.

Cracks were also found on angle sections that connect transversal joist to the truss nodes. An in depth investigation using liquid penetration tests helped to identify cracks in 70% of these connections. During geometric survey, it was detected a rotation of 1.5° on the pendulum bearings that supports the cracked transversal joist of the extremity. This rotation means that a longitudinal displacement of 18mm of the superstructure of bridge had occurred.

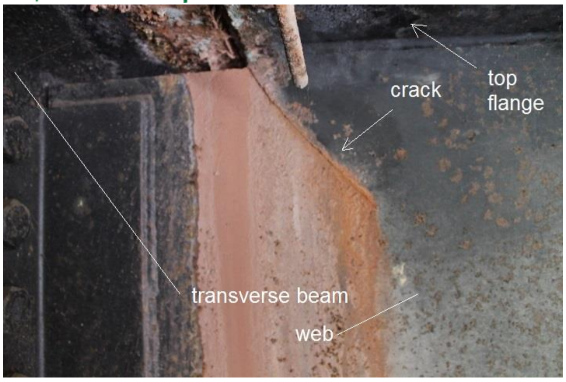


Figure 3 - Typical crack identified on a longitudinal joist of the bridge.

The advanced level of damage and wide-range of the cracks made obligatory that an emergency procedure was adopted, including reduction of train speed and study of mechanisms of crack formation to provide an emergency action and to assist a reinforcement design.

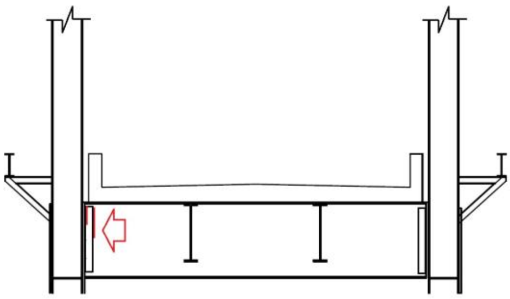


Figure 4 - Typical crack identified on the angled connectors of transversal joists and truss nodes.

The emergency action defined was to restore all the cracked elements welding the two cracked parts. Immediately the operational speed of the trains during passage by the bridge was reduced to 20km/h, remaining this way until finishing welding of the cracked elements.

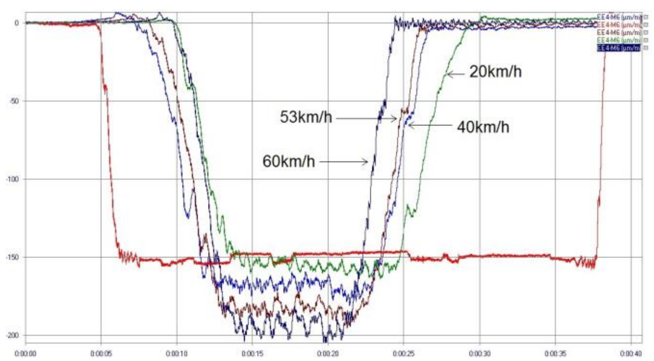
It must be emphasized that the reduced speed brings undesirable delays to the iron one transportation. Thus, a study to increase train speed was developed. For this study, data collected during dynamic tests as well as finite element models simulations were accomplished as can be seen in the sections that follow.

Mechanical properties of the steel

Assembly design specifies A588 steel to the structural elements of the bridge. To confirm these properties, a steel sample was extracted from the transversal joist of the extremity, the one in advanced cracked condition. Chemical and metallographic tests confirmed that composition was from a weathering steel. Measurements of Brinell hardness was around 185 BHn, from which could be defined that mechanical resistance was from an ASTM A588 "Corten" steel (yield strength of 345MPa).

Dynamic tests

Dynamic tests were accomplished on bridge structure before of the cracks' identification. To obtain the modal properties of the bridge, modal identification techniques based on frequency response functions (FRFs) were applied (MAIA and SILVA 1997). An instrumented train was used as excitation force to the structure. This train was composed with sixteen GDU gondola wagons. From this series of wagons the heaviest wagon is 145.7 tonnes, that spread onto four axles results in a 36.4tonnes/axle. This load level is in accordance with loads frequently transported by the railway. The bridge was instrumented with servo-accelerometers installed in the top and bottom chords of main trusses of the structure. Besides that, strain gages were installed at mid-span of top and bottom chords, and at end posts. Laser detectors were placed in the bridge entrances to register the velocity of the train's passages.



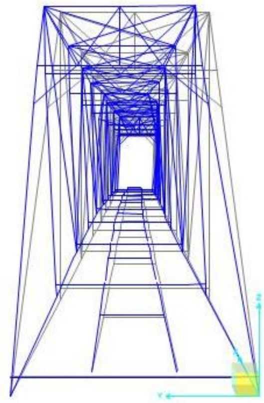


Figure 5 e 6 - Comparison of measured strains due to the train's passage with different speeds to the determination of DAF and Structural Model F.E.M

During the tests, the instrumented train passed over the bridge with speeds of 20km/h, 40km/h, 53km/h and 64km/h. Also the train stopped on the bridge to measure the static effects. From structural responses, dynamic amplification factors were determined for each train speed as presented on Figure 5. Maximum DAF's of 1.1, 1.22, 1.32 and 1.38 were determined for the speeds of 20km/h, 40km/h, 53km/h and 64km/h respectively.

From dynamic tests, the three first modal shapes were determined on frequencies of 1.30Hz, 2.34Hz and 3.66Hz, Figure 6.

After crack detection, an instrumentation was proposed with the aim of helping to decide which emergency procedure would be more efficient to restore structural bridge elements in the shortest possible amount of time. This way strain rosettes 0-45-90 were installed on the extremities of the cracks, on 3 situations, (A) crack without treatment, (B) crack with a 10mm hole on the extremity and (C) welded crack. Another instrumentation was adopted to evaluate interaction between longitudinal joists and concrete decks (concrete boxes). With this aim, top and bottom flanges of the I section longitudinal joist were instrumented with strain gages.

Finite element models

To evaluate bridge behavior a finite element model was developed using CSIBridge software. The geometry of the model was considered from the data provided by the geometric survey accomplished on the bridge. In this model I section beams were represented by frame elements, concrete boxes with shell elements and the railway tracks with nonlinear elements.

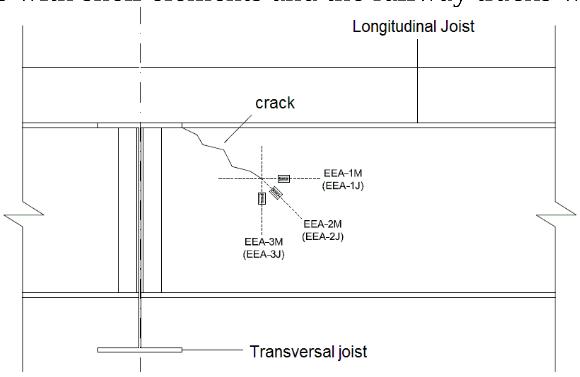


Figure 7 - Strain rosette installed on the longitudinal joist web to define corrective actions.

To calibrate the F.E.M. model, modal shapes identified during dynamic tests as well as static and dynamic effects due to train were compared, Table 1.

Table 1 - Comparison between calculated on calibrated model values and experimental values.

Variable	F.E.M.	Experimental
Modal shapes		
1^{st} natural frequency	1.29Hz	1.30Hz
2^{nd} natural frequency	2.38Hz	2.34Hz
3^{rd} natural frequency	2.67Hz	2.66Hz
Max static stress		
End post 3	47.82MPa	50.60MPa
End post 4	48.23MPa	51.65MPa
Top chord	-68.10MPa	-56.10MPa
Bottom cord	47.80MPa	40.30MPa
Max peak-to-peak acceleration (V=40km/h)		
Acceleration x*	158mg	97.7mg
Acceleration y	153mg	187mg
Acceleration z **	151mg	159mg
Max peak-to-peak acceleration (V=53km/h)		
Acceleration x*	163mg	109.5mg
Acceleration y	156mg	209mg
Acceleration z **	158mg	192mg

* Direction parallel to bridge axle.
** Direction vertical.

First static strains due to train stopped on the bridge were compared with strains calculated on F.E.M. for the calibration of the numeric model. With this aim, strains on end posts, bottom and top chords were measured using six strain gages in each transverse section of these elements, what made possible to measure bending and normal effects. In the F.E.M. model, enveloping values of stresses due to loads and train axle positioning in more unfavorable static effects were calculated to be compared with experimental values, reaching a good correlation between them (maximum difference of 17%). To evaluate dynamic effects train passages with speeds of 40km/h and 53km/h were simulated on F.E.M. model and accelerations in the middle span of the bridge were calculated. For a better adjustment of the transversal accelerations, it was necessary to consider 10% of vertical load in the transversal direction. It must be emphasized that Brazilian Standard recommends to consider 20%.

After calibration of the model, a detailed F.E.M. model was developed at this time using shell elements. The details of the connection between longitudinal and transversal joists were represented in the model to study stress concentration factors in this region, where cracks were identified. The interaction of this model with main trusses was considered with translational and rotational springs imposed on the extremities of the transversal joist. Stiffness's of these springs were calculated on calibrated F.E.M. model. Dead and train loads were applied in the detailed F.E.M. model. Stresses on longitudinal joists were calculated and are presented on Figure 8.

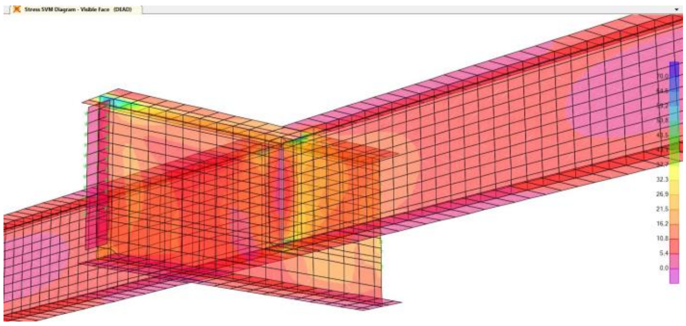


Figure 8 - Von Mises stresses in the region of connection between longitudinal and transversal joists due to train load.

It may be noticed in Figure 8 that the stress concentration occurs on the longitudinal joists, with a sharp edge in its web in the previous space where the angle makes connection with transversal joist. In this region concentration factors around 1.7 were calculated in the detailed F.E.M. model. Stresses due to dead load and train load were calculated 65MPa and 250MPa respectively. Therefore, the stress sum is equal to 315MPa, close to the yielding limit of the steel (345MPa for a ASTM A588 steel).

In the region of connection between transversal joist and truss node, stresses due to dead load and train load were calculated 70MPa and 40MPa respectively, a good way off from yielding limit of steel. So cracks identified on these regions could not be explained with this stress levels. For a better investigation, stresses were recalculated considering cracking of longitudinal joists, that could rearrange load distribution over the joists that forms the grillage support structure. The results indicated that an increase of only 5% in stresses due to load rearrangement is reasonable, which is not sufficient to explain the cracks. In the following, simulations with calibrated F.E.M. model made clear that stresses due to the function of the transversal joist as a bracing to equilibrate planar transversal frames were being superposed with dead and trains strains and causing the cracks.

24 hours Monitoring

To evaluate thermal effects on bridge structure, a 24 hours continuous monitoring was accomplished. During monitoring, temperature variation was 12.6°C, with minimum of 21.7°C and maximum of 34.3°C. Temporal series of strain are presented on Figure 9. In this case, mean of strains were calculated hourly, in the break when there were not train on rail.

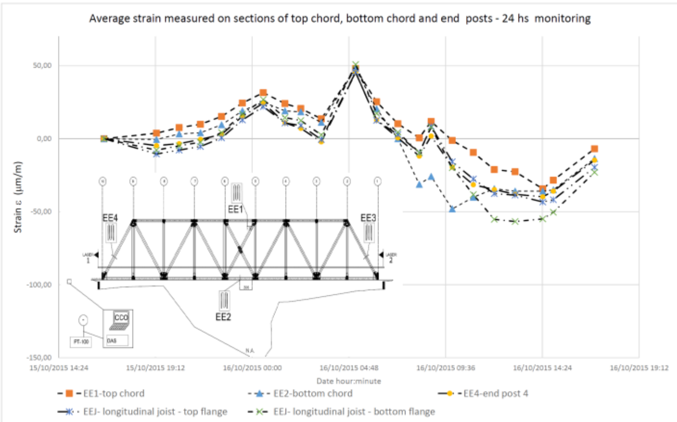


Figure 9 - Temporal series of strain measured on sections of top chord, bottom chord, end post and longitudinal joist.

Maxima strain variation due to temperature variation were 130.4µm/m and 235.0µm/m respectively on top and bottom chord, 152.8µm/m and 151.9µm/m on end posts and 121.5µm/m on the flanges of longitudinal joist.

Bolt preload verification

During bridge assessment, preloads on black bolts of the bridge connections were measured using a torque transducer. For bolts with preloads smaller than 70% of the bolt tension strength, a torque wrench was used to tighten the bolt. It was verified that most of the loose bolts were in the connection of the two transversal joists of the extremity. Dynamic measurements also indicates that higher impacts during the train passage occurs in these two elements, which can explain these looseness of bolts.

Stresses around cracks

Direction of principal strains were calculated from data registered by strain rosettes installed on extremity of the cracks identified on web of longitudinal joist. Also these directions were calculated on detailed F.E.M. model as presented in Figure 10. Comparison between theoretical and experimental values is presented on Table 2.

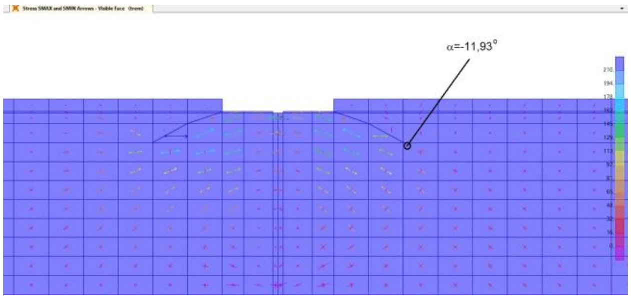


Figure 10 - Direction of principal strains around cracks on the web of longitudinal joists.

Reinforcement design

Even though welding of cracked joists was applied as an emergency solution, from F.E.M. model analysis it was clear that stresses concentration on the web longitudinal joists would lead to crack formation soon. To avoid cracks on longitudinal joists, a reinforcement design was developed taking as target a better load distribution on the under-track structure, see Figure 11. With this aim I beams were proposed to connect longitudinal and transversal joists. Simulations in the F.E.M. model has confirmed that this reinforcement would make stresses in the sharp edge region decrease about 20%.

To avoid cracks in the on angle sections that connects transversal joist to the truss nodes, longitudinal stiffeners were proposed to be welded over the angled connectors.

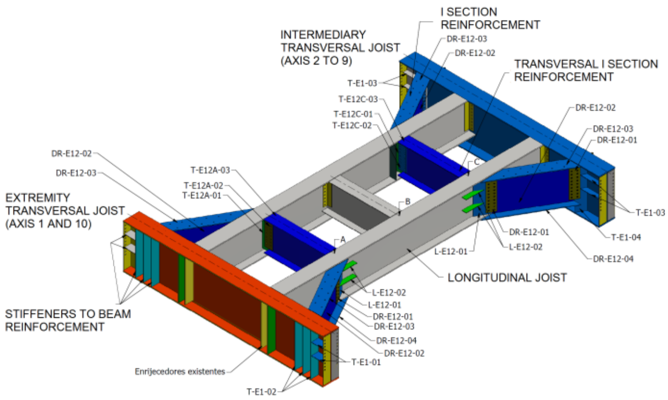


Figure 11 - Reinforcement proposed to under-track structure of the Fumaça Waterfall Bridge.

Remaining-Life of the Bridge - Fatigue analysis

Remaining-life of structural elements of the bridge was calculated by a fatigue analysis. With this aim a hybrid analysis using data from monitoring and from F.E.M. models was accomplished. Static stresses due to dead load was calculated on the F.E.M. model. Static stresses due to temperature variation were estimated from 24hs monitoring data, adopting a regular variation of 15°C over any one day. Cycles caused by train passages were also considered from monitoring data. From 24hs-monitoring data, daily traffic of 25 trains was considered during fatigue analysis.

In this study, fatigue strength of steel was defined by S-N Wohler curves in accordance with parameter curves specified by American Association of Railroads (AAR). A Rainflow analysis was first accomplished to count the stress cycles induced by train, Figure 12. Remaining-life for truss elements was calculated over 100-years.

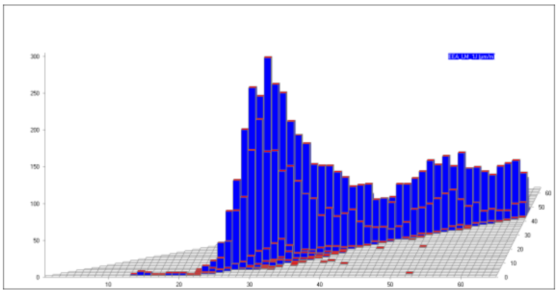


Figure 12 - Rainflow analysis to stresses caused by train passage on 40km/h, sharp edge region of the longitudinal joist.

Fatigue analysis of the joist elements that forms the under-track structure was accomplished taking into account also welding of cracks provided in emergency actions. To consider cycle-stresses due to train's passages, data collected on strain rosettes were considered, using a correction factor of 2. This factor was defined on F.E.M. detailed models to compensate the position of the rosettes that were installed in the extremity of the crack and not on the region of sharp edge of the longitudinal joist, where concentration stress is more pronounced. To help railway administrator in the decision of limiting speeds, remaining-life for the joists were calculated for three operational speeds, 40km/h, 53km/h and 64km/h. Results for this analysis is presented in Figure 13. For the reinforced structure, where a 20% of reduction of stresses in the sharp edge region of longitudinal joist is expected, remaining-life over 100-years was calculated.

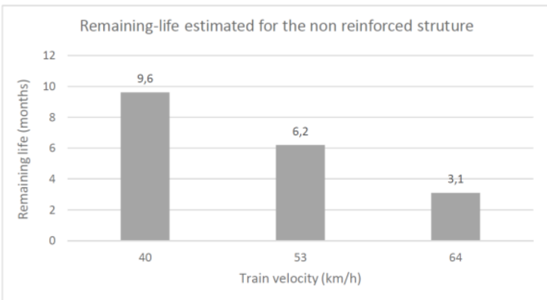


Figure 13 - Remaining-life calculated for the non reinforced structure.

From the results of fatigue analysis, the concessionaire of the railway decided to limit the train velocity to 40km/h, which is considered acceptable by operational division of the concessionaire. Currently mobilization to execute reinforcement is being provided by M.R.S. considering the target limit time of execution of six months.

Conclusions

During structural assessment of the Fumaça Waterfall Bridge, a systematic cracking scenario was identified in the joists that compose the under-track structure of the bridge. Faced with the advanced cracking scenario, emergency procedures were adopted, including reduction of train's speed, study of mechanisms of crack formation to provide an emergency action and to assist a reinforcement design. From the monitoring and simulations performed with finite element models, it was concluded that cracks identified on the web chamfer of longitudinal joists, close to the connection with transversal joists, is due to details mistakes. In these elements, continuous-beam behavior leads to an increase of the bending moment, causing in the sharp edge region stresses closer to steel strength.

Cracks identified on angle sections that connects transversal joist to the truss nodes can be explained by the superposition of stresses caused by torsion and bending and also from the frame behavior where the transversal joist acts as a bracing to equilibrate planar transversal frame. In depth inspection of the cracks helped to decide that welding all the cracks would rehabilitate the bridge for a special condition of utilization, during a sufficient time until a definitive rehabilitation could be provided. To define this condition of utilization the remaining-life of structure was calculated considering operational scenarios of velocity of the train. From these analysis, the velocity of 40km/h was defined as the one which is regular for the operation of the railway and which enables to provide an appropriate reinforcement. A reinforcement design was elaborated taking as target a better load distribution on the under-track structure.

From fatigue analysis, it was concluded that this reinforcement restores a remaining-life of 100-years to this structure, which is one of the strategic bridges of the railway.

References

CEN. European Committee for Standardization 2003.

CSI BRIDGE. Structural and Earthquake engineering software. 2016.

ABNT, NBR8800. Projeto de estruturas de aço, estruturas mistas de aço e concreto de edifícios. 2008.

AAR. Association of American Railroads. 2009.

MAIA, N.M.; SILVA, J.M. Theoretical and experimental modal analysis. 1997

RODRIGUES, J.F.; CASAS, J.R.; ALMEIDA P.A. Fatigue-Safety assessment of RC bridges. 2011.

APPLICATION TO THE BRAZILIAN HIGHWAY NETWORK. Structure and Infrastructure Engineering. 2013.

SILVA, J.M.; MAIA, N.M. Modal analysis and testing. 1999.