ATLSS Reports | Civil and Environmental Engineering

9-22-2010

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FINAL REPORT

on

FHWA CONTRACT No. DTFH61-07-C-00007

"IMPROVED CORROSION RESISTANT STEEL FOR HIGHWAY BRIDGE CONSTRUCTION"

by

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ATLSS Report No. 10-11

September 22, 2010

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ABSTRACT

Highway Bridges are normally fabricated from steels specified in ASTM 709, "Standard Specification for Structural Steels for Bridges". The steels range in minimum yield strength from 36ksi, (A36) to 100 ksi, (A514). The preferred steel is A588, minimum yield strength of 50 ksi, because it forms an adherent corrosion-resistant oxide film under ambient conditions when it is alternately wetted and dried. However, it is not corrosion resistant when it is exposed to the salt solution that develops when the bridge is covered with snow that is treated with deicing salt or when the bridge is close to the ocean and exposed to salt spray. Therefore, the Federal Highway Administration (FHWA) contracted with the Lehigh University Center for Advanced Technology for Large Structural Systems, (ATLSS) to develop "an improved corrosion-resistant steel for Highway Bridge Construction". FHWA required that the improved steel (1) could be produced by American steel companies on existing facilities, (2) would meet existing AASHTO design specifications, (3) have a life-cycle cost less than A709 steels, (4) have mechanical properties similar to today's steel grades", (5) be readily weldable by SAW, SMAW, FCAW, and GMAW, and (6) have a cost similar to A709 steels.

ATLSS evaluated 23 developmental steels involving the addition of Cu, Ni, Si, or Cr to a Cu-Ni precipitation-strengthened steel that was similar to an HPS-100W steel developed by ATLSS for infrastructure applications, primarily bridges and that had been fabricated into a number of bridges. Through control of the aging temperature, the yield strength of the steel could be varied from 60 to 100ksi, with a preference of 70 to 80ksi for the highway bridge application. The results of the corrosion tests on coupons exposed to 5% NaCl indicated that the best steel was a 2%Cu-2%Ni steel (coded Steel D).. This steel along with three other "good" steels were exposed to 1% and 3% NaCl along with A36 and A588 benchmark steels. The results again indicated the superiority of Steel D at both additional NaCl concentrations. With respect to the other FHWA requirements, (1) Steel D is similar to the 100W steels that were previously produced in the USA, (2) it can be readily designed to AASHTO specifications, (3) a detailed life-cycle-cost study indicated that it's life-cycle cost is better than that of A588 after ten years and for all years thereafter, (4) it has excellent ductility and toughness superior to most of today's steel grades, (5) in the slow-notch-bend weldability test, its performance was excellent, and (6) its cost is slightly higher than most A709 steels but that is more than offset by its improved life-cycle cost and its outstanding mechanical properties and weldability.

Production of a commercial heat of Steel D is recommended to (1) confirm its excellent mechanical properties and weldability, (2) conduct large-scale prototype tests, and (3) retain sufficient steel slabs that can be rolled to the structural components for fabricating and erecting three highway bridges at locations selected by FHWA where improved corrosion resistance is necessary to assure safe long-term operation

BACKGROUND AND INTRODUCTION

Currently, steels approved for the fabrication and erection of bridges are specified by the American Society for Testing and Materials (ASTM) in ASTM 709, "Standard Specification for Structural Steel for Bridges" as follows:

ASTM Specification	Minimum Yield Strength, ksi
A36	36
A588	50
A852	70
A514	100

The most widely used steel for bridges is A588 because it forms a relatively impervious oxide patina upon ambient alternate wetting and drying and thus provides an effective desirable life-cycle cost. The higher-strength steels are also attractive because the plate thickness used in fabricating structural sections can be significantly reduced, particularly for the 100-ksi minimum-yield-strength steels. However, all the steels, including A588 are susceptible to significant corrosion in those regions of the country where snow fall is heavy and large amounts of sodium chloride (NaCl) and or calcium chloride ($CaCl_2$) are used to minimize the dangers of ice and snow on bridge roadbeds. In addition, all 13 grades of A514 require preheat during welding to avoid hard brittle heat-affected-zones and do not exhibit a high level of fracture toughness.

Over an extended period, the Lehigh University Center for Advanced Technology for Large Structural Systems (ATLSS) developed[1to10**] a Cu-Ni precipitation-strengthened infrastructure steel, primarily for bridges, that exhibits excellent toughness and weldability as a result of markedly lowering the carbon content and offsetting the loss in strength through the precipitation of strengthening coherent copper particles. Thus all development steels subsequently investigated were of the Cu-Ni precipitation type. (** See References)

HISTORY

The success of American commerce and industry depends significantly on an available and efficient transportation infrastructure, including railroads, highways, and waterways. The efficiency of these transportation elements is greatly enhanced by bridges at the elemental intersections, and the need for bridges has greatly increased since the 1950s as a result of the continued expansion of the interstate highway system. Correspondingly, the cost of maintaining, rehabilitating, and replacing bridges is now at the trillion-dollar level. In Pennsylvania, the state with the largest number of bridges, the cost of the preceding requirements has resulted in the Governor requesting extensive new state taxes exclusively for bridges, which are in a generally deplorable condition. Federal oversight of bridges resides with the federal Department of Transportation and its Federal Highway Administration (FHWA). At the state level, responsibility resides with the various Departments of Transportation, who cooperate in many areas as the American Association of State Highway Transportation Officials (AASHTO).

Bridges are constructed from reinforced concrete or steel with steel being the dominant material because of design and fabrication flexibility. Material specifications for steel bridges are promulgated by the American Society for Testing and Materials (ASTM) in Designation: A709, "Standard Specifications for Structural Steel for Bridges", which has also been adopted by AASHTO. The specification covers minimum yield-point strengths, in ksi (MPa) of 36 (250), 50 (345), 70 (485), and 90 (620) in plate thicknesses through 4 inches (102 mm), and 100 (690) through 2-1/2 inches (64 mm).

In recent years, significant advances have been made in the melting and solidification practices for producing steel, including metallurgical-ladle treatment and continuous-casting. This advance has resulted in quality improvements and the designation of such treated steels as High-Performance Steels (HPS), properties that permit their use as fracture-critical bridge members. These improved HPS steels are expected to eliminate many of the types of failure described in J.W.Fishers' book "Fatigue and Fracture in Steel Bridges". Research involving the development and application of HPS has been carried out at several university and industrial laboratories. At the Lehigh University Center of Advanced Technology for Large Structural Systems (ATLSS), founded by Dr. Fisher, research has included metallurgical development of new HPS compositions at yield strengths of 70- and 100-ksi (HPS-70W and -100W), production of commercial heats, and the fabrication and testing of large bridge element prototypes. The metallurgical improvement in toughness and weldability of these steels and their excellent performance in structural tests has resulted in improved design and fabrication of bridges at these increased strength levels, and the improvement has been confirmed in tests at the Turner-Fairbanks Laboratory of FHWA.

The improvement in the application of these steels to bridges has not been reflected significantly in extended life cycles, and therefore, in the cost of repair, maintenance, and replacement, particularly of highway bridges because of the corrosive effect of the salts used to deice bridges in the many locales where snow and ice impact transportation. Therefore, FHWA issued a Request for Proposals to develop an "Improved Corrosion Resistant Steel for Highway Bridge Construction". The Lehigh ATLSS Center submitted a proposal[11] and was selected to undertake that investigation.

FHWA required that the steel to be developed conform to the following:

1. Be Producible by American Steel companies on existing facilities
2. Meet AASHTO design specifications
3. Have a life-cycle cost less than A709 steels
4. Have mechanical properties similar to "today's steel grades"
5. Be Readily weldable by SAW, SMAW, FCAW, and GMAW
6. Have a cost similar to A709 steels

EXPERIMENTAL PROCEDURE

Melting and rolling

During the experimental program, twenty three developmental steels were melted and rolled by the U.S. Steel Technical Center to ATLSS specifications Their chemical compositions involved variation of the alloying elements in the previously noted Cu-Ni precipitation-strengthened steel, whose base composition was as follows:

Mn	Cu	Ni	Cr	Mo	V	Cb
1.00	1.20	0.75	0.50	0.50	0.06	0.02

Twenty one developmental steels were melted as 300-pound heats and the heat was adjusted to three statistically related compositions of 100-pounds each, cast, and the ingots were then rolled to 1-inch-thick plate. Half of the 1-inch plate was retained as 1-inch plate for subsequent heat-treatment and mechanical-property testing by ATLSS. The other half was reheated and rolled to 0.1-inch-thick sheet for subsequent corrosion testing by Dr. D.C.Cook's Advanced Metal Coating Analysis Company and by Ms. Megan Conrad at Lehigh University. The chemical composition of these steels is listed as the first twenty-one steels in Table I. Two additional 500-pound heats (Steels 22 and 23 coded as Steels SS and DD) of compositions believed to be promising were melted and rolled to 1-inch-thick plate for heat treatment followed by mechanical-property and weldability testing and into as-rolled 0.1-inch-thick sheet for corrosion testing.

Mechanical-Property Testing – The mechanical-property tests were conducted in accordance with ASTM SE8-94a- Test Method for Tension Testing of Metallic Materials and ASTM SE 23-94A - Test Method for Notched Bar Impact Testing of Metallic Materials.

Weldability Testing Weldability testing was based on the Lehigh Slow-Notch-Bend Test illustrated in Figure 1. The test was modified by adding a second parallel 4-inch long weld bead so that the two weld beads touched each other and were centered on the width of the 3-inch wide specimen, which was 12-inches long, notched to a depth of 0.050 inch below the surface using a Charpy V-notch cutter producing a root radius of 0.001 inch and tested by center-length loading on an 11-inch span. The vertical deflection was increased until fracture occurred or until the loading platen deflected a full 1 inch. The test was repeated with multiple specimens at progressively lower temperatures. The lowest temperature at which the full vertical deflection of 1 inch was obtained without propagating a fracture was the measure of weldability. Obtaining the full 1-inch deflection at a temperature of -120F was considered "excellent", at -80F "very good", at -40F "good", at 0F "fair", and above 0F as "poor".

Salt Spray Chamber Testing Procedure by Advanced Metal Coating Analysis -The resistance of the developmental steels to corrosion in highway bridges was measured by the mass-loss (weight loss) of cleaned samples exposed to an environment simulating the corrosive environment of highway bridges in the presence of deicing-salt solutions. The primary method of replicating the corrosive environment utilized accelerated cyclic corrosion tests (CCT). This testing procedure was necessary to reduce the exposure time required to observe significant corrosion under normal atmospheric conditions to a time that permits meaningful data to be obtained on the proposed modified developmental steel compositions in two years. The exposure conditions used for CCT duplicated as closely as possible those of highway bridges, such as the Moore Drive Bridge in Rochester, New York, and the Kure Beach 25 meter test lot, (KB25m). Both locations have a corrosivity of C5, the highest category permitted by ASTM. Samples from both sites were used to confirm as much as possible the effectiveness of the accelerated cyclic-corrosion tests. A sequence of developmental steels was formulated so that, at the earliest possible time, the effect of composition could be analyzed, comparatively and statistically, and other of the formulated steel compositions or entirely new compositions could be produced and corrosion tested in rapid succession. Supplementing the mass-loss data were spectroscopic analyses of the rusts formed on the test samples to provide additional information on the effect of the chemical elements providing the best resistance to corrosive attack. The results of all the corrosion tests were compared with corresponding test data on benchmark steels, Appendix I, ranging from low-cost steels known to be subject to corrosion (A36) to steels with excellent resistance but relatively high cost A1010. A588 is the most commonly used steel in highway bridges. Each run cycle for CCT consisted of seven types of steels, the three developmental steels and four well studied standards. The corrosion data for the developmental steels will not only provide absolute corrosion rates, but also relative corrosion rates to known standards. All corrosion testing and evaluation were performed according to ASTM standard testing guidelines. The tests were originally conducted at a salt-spray concentration of 5%NaCl. On the basis that 5% may be too aggressive, tests on six of the best steels at 5% and A588 were repeated at 1% and 3% NaCl.

Electrochemical Surface Potential and Mass Loss Corrosion Investigation of Improved Corrosion Resistant Steels for Highway Bridge Construction

The accelerated cyclic corrosion protocol utilized 5% NaCl solution for coupon immersion (SAE J2334 Standard Procedure). The steel coupons at Lehigh University were subjected to a 15 minute electrolyte immersion on week days unlike the electrolyte exposure protocol used by Advanced Metal Coating, which involved an electrolyte spray method for 15 minutes daily. The developmental steels being studied were Steels 12, D, M, R, S, T, and Y along with the A588 weathering benchmark steel. While the electrochemical surface potential approach was determined experimentally to be a valid procedure to measure the corrosion rate of steel at zero and one week exposure intervals, the five week exposure samples were too heavily corroded to attain meaningful potential and current values. Overall, four different steels were tested after five weeks exposure and inadequate electrochemical data were collected for all four compositions such that no meaningful corrosion information was obtained. Thus, electrochemical surface potential measurements were not continued on steel samples at or beyond the five week exposure period. Instead, a revised procedure was adopted to add additional steel coupons for each of the eight steels to the cyclic corrosion protocol such that the corrosion rate could be calculated using electrochemical surface potential measurements at shorter exposure lengths of up to two and three weeks. The ideal sample for electrochemical measurements, according to Courtney Neel at Princeton Applied Research, is a “metallic conductor that is uniform in surface chemistry and free of oxides.” Therefore, the electrochemical data collected on the bare steel at 0 weeks provided the most meaningful evidence of the effects of elemental additions on the corrosion tendencies of steels.

Mass loss measurements were utilized to calculate the corrosion rate of the eight developmental steels at 1,5,10, and 14 week exposure intervals. These measurements were carried out for correlation purposes by means of chemical-stripping analysis as (ASTM G1-90, Cleaning Designation C 3.5) identical to that employed by Advanced Metal Coating The ultimate goal was to determine any correlations between the corrosion data collected at Lehigh University from both the electrochemical measurements and the mass loss experiments with the data obtained by Advanced Metal Coating.

EXPERIMENTAL RESULTS

The 23 developmental steels included in the experimental program were evaluated and the information reported in accordance with the following outline:

Testing Sequence by Steels

Report No.	Dates	Report Title	Steels Evaluated
0	9/1/06 - 12/31/06	Preliminary Evaluation Report	9,10,12
1	1/1/07 - 3/31/07	Quarterly Report No. 1	J,K,M
2	4/1/07 - 6/31-07	Quarterly Report No. 2	E,F,H
3	7/1/07 - 12/31/07	Quarterly Report No. 3	A,B,D
4	1/1/08 - 3/31/08	Quarterly Report No. 4	A,B,D
5	4/1/08 - 6/31/08	Quarterly Report No. 5	U,V,W
6	7/1/08 - 9/31/08	Quarterly Report No. 6	R,S,T
7	10/1/08 -12/31/08	Quarterly Report No. 7	X,Y,Z
8	1/1/09 - 3/ 31/09	Quarterly Report No. 8	SS-500#
9	4/1/09 - 6/31/09	Quarterly Report No. 9	DD-500#
10	7/01/09-12-31-10	Quarterly Report No. 10	A588

The changes in composition from that of the base steel are summarized in Table II. The rationale for the changes was as follows:

Report 0/Steels 9,10,12 – The most important composition change was an increase in the chromium content from the standard 0.50 % to 4% in Steel 12, which significantly improved corrosion resistance but embrittled the steel, so that ductility and toughness were extremely poor and welding can be expected to require preheat. Thus the high-hardenability, strengthening carbide-forming elements, Cr, Mo, and V were removed.

Report 1/Steels J,K,M – Eliminating the carbide-forming elements, particularly the chromium, resulted in excellent ductility and toughness and expected good weldability. In addition, when the copper and nickel in were increased to nominally 2% in Steel M significantly improved corrosion resistance was observed.

Report 2/Steels E,F,H – Increasing the silicon content to 2% did not significantly improve the corrosion resistance and embrittled the steels.

Report 3&4/Steels – A,B,D – Increasing both copper and nickel to 2% in Steel D resulted in excellent corrosion resistance for Steel D and reducing the carbon content to nominally 0.025% resulted in excellent ductility and toughness for all three steels as illustrated in Table III and excellent weldability as subsequently discussed.

Report 5/Steels U,V,W – Increasing the silicon to 3% embrittled all three steels.

Report 6/Steels R,S,T – Increasing chromium from 2% (Steel R) to 4%-(Steel S) significantly improved the corrosion resistance, but a further increase to 6%-(Steel T) resulted in a change in

the character of the corrosion product and poorer corrosion resistance. Moreover, chromium at or above 2% embrittled the steels.
Report 7/Steels X,Y,Z – Reducing the nickel to 0.5% and increasing the chromium to 2% or 4% did not improve the corrosion resistance and embrittled the steels.

Report 8/Steel SS – The slow-notch-bend tests confirmed the deleterious effect of chromium on weldability. The 1-inch notch-bend requirement occurred between 0 and 20F for Steel SS, considered to be poor performance.

Report 9/Steel DD-The desired 1-inch notch-bend behavior was observed down to -100F for Steel DD (D) considered to be excellent performance.

Report 10/Steel A588-The 1-inch notch-bend requirement occurred at 0F for Steel A588, considered to be fair performance, but borderline for requiring preheat for welding.

DISCUSSION OF RESULTS

The importance of the results obtained on the 23 developmental steels can best be assessed by relating them to the following previously identified FHWA requirements:

1. All the developmental steels can be readily produced on existing American Steel Company facilities. Several heats of the precipitation-strengthened Cu-Ni HPS 100W steel have been produced in the U.S. with no production problems.

2. Several bridges fabricated from HPS 100W steel have been designed to AASHTO specifications and fabricated and erected in the U.S. with no reported problems. Similar design characteristics are anticipated with respect to those developmental steels that are recommended for highway-bridge applications, because their base composition is similar to HPS 100W in that they are precipitation-strengthened low-carbon steels.

3. The life-cycle cost for the most promising developmental steel, Steel D, is compared in Appendix II with that of A588, the most commonly employed A709 bridge steel. The comparison suggests that Steel D is twice as good as A588 from the corrosion standpoint and that its life-cycle cost is better than A588 after ten years and for all years thereafter. These results are consistent from the corrosion standpoint with results recently reported by Nippon Steel in "Nippon Steel News, No. 377, March 2010". The report states that U.S.Steel's COR-TEN steel adopted by ASTM as A588 performs very well in inland locations but not very well in seaside locations exposed to constant NaCl spray. To improve seaside performance, they eliminated the chromium and added 3% nickel to COR-TEN steel to produce " NAW-TEN" steel, a nickel-added advanced weathering version of COR-TEN steel. We have no basis for comparing Steel D with NAW-TEN but at the usual 0.15% carbon content for COR-TEN steel, the ductility, toughness, and weldability would be expected to be substantially inferior to Steel D.

4. The mechanical properties of most of the developmental steels were "similar to those of today's steel grades", as listed in Table III for Steels A, B, and D, which illustrate the desirable

outstanding ductility and fracture toughness of these low-carbon precipitation-strengthened steels whose yield strength could be adjusted from 60 to 100 ksi by aging at temperature from 1250F to 1000F. Several attempts were made to investigate the potential of chromium additions despite the brittleness encountered with Steel 12(4%CR) because of its well known corrosion resistance in stainless steels. This included Steels V,W,R,S,T,X,Y,and Z with chromium contents from 2 to 6%. In all cases, the mechanical properties were relatively poor compared with Steels A,B,D, and their corrosion resistance was poorer than Steel D. The same result was observed for high silicon steels.

5. The developmental steels identified as promising can be readily welded by SAW, SMAW, FCAW, and GMAW processes. Specifically excluded are the steels with significantly increased chromium because they would probably require preheat. The slow-notch-bend weldability tests indicated that Steel D (DD) arrested crack propagation down to -100F whereas A588 (5N) did so to 0F, and Steel S (SS) to +35F. These results suggest that martenstic steel weldments containing significant chromium additions may rapidly propagate cracks formed as a result of welding.

6. The base cost of all the developmental steels are higher than those of the least costly A709 steels because the alloy additions increase the cost above that for A36 or A588 steels, but the life-cycle cost was found to be an appropriate offset.

The corrosion characteristics of the developmental steels are described in Appendix I and the respective mass-loss values of the five "best" steels compared with the A588 and A36 benchmark steels after exposure for 70 days in the salt-spray cabinet at 1%, 3% or 5% NaCl were as follows:

Steel Thickness Loss in microns after Exposure at Indicated NaCl Concentration

	1%	3%	5%
DD (2%Ni+2%Cu)	430	679	550
M (2%Ni+2%Cu)	436	687	577
SS (4%Cr)	833	737	667
Y (1%Cu+4%Cr)	1000	834	666
R (2% Cr)	1238	1199	677
5N (A588)	1037	1250	910
A36	1134	1400	1078

CONCLUSIONS

Under contract to the Federal Highway Administration (FHWA), the Lehigh University Center for Advanced Technology for Large Structural Systems (ATLSS) implemented a program to develop an improved corrosion-resistant highway-bridge steel. The contract required that the steel be producible on existing American facilities, meet AASHTO design specifications, with mechanical properties similar to "today's steel grades", be readily weldable by standard processes, and have a cost similar to A709 steels.

Twenty-three developmental steel compositions were evaluated with elemental additions to a base Cu-Ni precipitation-strengthened steel developed by ATLSS that was previously shown to have excellent toughness and weldability for infrastructure applications, particularly bridges.

Various amounts of copper, nickel, chromium, and silicon were added to the base composition. The steel that best met the FHWA requirements was identified as Steel D, which contained 2% copper and 2% nickel. Several steels with increased chromium exhibited good corrosion resistance but the other properties, particularly toughness and weldability, were not acceptable.

Production of a commercial heat of Steel D is recommended for (1) confirming its excellent mechanical properties and weldability, (2) conducting large-scale prototype tests, and (3) for retaining steel slabs that can be rolled to the structural components required for erecting three highway bridges at locations selected by FHWA where improved corrosion resistance is highly desirable.

ACKNOWLEDGEMENTS

1. The Lehigh University ATLSS Center wishes to acknowledge the financial support and technical guidance of the Federal Highway Administration for FHWA Contract No. DTFH61-07-00007, "Improved Corrosion Resistant Steel for highway Bridge Construction".
2. The efficient and precise melting and processing of the 23 developmental steels by the U.S. Steel Technical Center was crucial to the undertaking of the program.

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TABLES I, II and III

Table I.
Chemical Composition of Developmental Steels, %

Steel	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	V	Cb	Al	CE
9	0.070	1.46	0,012	0.006	0.76	1.16	0.74	0.51	0.49	0.06	0.02	0.039	0.50
10	0.070	1.43	0.011	0.006	0.76	1.13	1.12	0.50	0.49	0.06	0.02	0.037	0.52
12	0.070	1.35	0,011	0.006	0.76	1.08	1.83	3.82	0,47	0.06	0.02	0.027	1.12
J	0.070	1.49	0.017	0.004	0.76	1.24	0.76	0.006	0.001	0.003	0.02	0.033	0.50
K	0.071	1.45	0.017	0.004	0.76	1.26	1.99	0.005	0.001	0.004	0.02	0.029	0.58
M	0.075	1.45	0.017	0.004	0.75	1.96	1.97	0.005	0.001	0.004	0.02	0.028	0.63
E	0.032	1.49	0.013	0.003	2.08	1.30	0.74	0.005	0.001	0.002	0.02	0.037	0.56
F	0.028	1.46	0.013	0.002	2.00	1.28	1.98	0.005	0.001	0.002	0.02	0.034	0.62
H	0.036	1.45	0.013	0.003	2.01	1.89	1.96	0.005	0.001	0.002	0.02	0.037	0.67
A	0.023	1.43	0.015	0.002	0.75	1.31	0.73	0.006	0.004	0.002	0.02	0.037	0.45
B	0.024	1.40	0.014	0.002	0.75	1.30	1.93	0.006	0.004	0.002	0.02	0.029	0.53
D	0.024	1.40	0.014	0.002	0.75	2.04	1.92	0.006	0.004	0.002	0.02	0.027	0.57
U	0.024	0.74	0.015	0.002	3.06	1.20	0.74	0.007	0.003	0.002	0.003	0.032	0.48
V	0.024	0.72	0.015	0.002	3.02	1.17	0.72	1.89	0.004	0.002	0.004	0.028	0.85
W	0.024	0.72	0.015	0.002	3.02	1.89	0.72	1.88	0.005	0.002	0.004	0.027	0.90
R	0.014	1.03	0.013	0.003	0.32	0.98	0.60	2.06	0.004	0.003	0.02	0.023	0.68
S	0.014	1.00	0.014	0.001	0.32	0.97	0.59	3.95	0.005	0.004	0.02	0.019	1.08
T	0.015	1.00	0.012	0.003	0.32	0.93	0.57	6.23	0.007	0.006	0.02	0.017	1.54
X	0.024	1.03	0.014	0.004	0.33	1.00	0.51	2.06	0.007	0.060	0.03	0.031	0.84
Y	0.024	1.01	0.013	0.004	0.76	0.96	0.50	4.28	0.008	0.060	0.03	0.025	1.28
Z	0.024	1.00	0.012	0.003	2.19	0.93	0.48	4.27	0.008	0.060	0.03	0.027	1.38
SS	0.039	1.28	0.009	0.002	0.27	1,02	0.75	4.10	0.010	0.005	0.02	0.023	1.30
DD	0.025	1.00	0.014	0.003	0.75	2.00	1.89	0.009	0.006	0.003	0.02	0.017	0.57

Table IA. Chemical Composition of Benchmark Steels, %

	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	V	Cb	Al	CE
A36	0.055	0.93	0.013	0.009	0.20	0.30	0.17	0.20	0.05	0.002	---	0.026	0.30
A588-1	0.120	1.14	0.014	0.012	0.36	0.31	0.30	0.52	0.003	0.037	---	0.056	0.49
A588-2	0.110	0.97	0.011	0.015	0.37	0.27	0.24	0.52	0.09	0.036	---	---	0.46
A588-5N	0.110	0.96	0.012	0.009	0.36	0.27	0.23	0.53	0.08	0.035		0.014	0.44
A852	0.091	1.26	0.017	0.008	0.37	0.30	0.30	0.56	0.10	0.060	---	0.016	0.51
HPS 100W	0.056	1.00	0.006	0.003	0.27	1.00	0.75	0.51	0.49	0.060	0.003	0.032	0.57
A1010	0.030	1.50	0.040	0.005	1.00	---	1.50	12.00	---	---	---	---	2.85

Table II. Sequence of Adjustments to Base Cn-Ni Precipitation-Strengthened Steel.

Steel Code	Elemental Addition
9	Base(B)
10	B + Ni(L12)
12	B +Ni(L83)+Cr(3.82)
J	B-Cr
K	B +Ni(1.99)-Cr
M	B + Cu(1.96)+Ni(1.97)-Cr
E	B + Si(2.08)-Cr
F	B+ Si(2.00)+Ni(1.98)-Cr
H	B + Si(2.01)+Cu(1.89)+Ni(1.96)-Cr
A	Base repeat-Cr
B	B+Ni(1.93)-Cr
D	B+Cu(2.04)+Ni(l.92)-Cr
U	B+Si(3.06)-Cr
V	B+Si(3.02)+Cr(1.89)
W	B+Si(3.02)+Co(1.89)+Cr(l.88)
R	B+Cr(2.06)
S	B+Cr(3.95)
T	B+Cr(6.23)
SS	Same as S - 500#heat
DD	Same as D - 500# heat

Note: Except for Steels 9,.10, and 12,Mo and V were removed and were present only as residuals.

Table II. Mechanical Properties for Steels A, B, and D*

C	Mn	P	S	Si	Cu	Ni	Cr	Mo	V	Cb	Al
0.023	1.43	0.015	0.002	0.75	1.31	0.73	0.006	0.004	0.002	0.02	0.037

		Tensile Properties					Hard	Charpy V-Notch Transition Temperature					Charpy V-Notch Energy				
		YS	TS	EL	RA	YS/TS	HRc	20 ft-lb	35 ft-lb	60 ft-lb	15 mils	50% FAT	70	0	-40	-80	-120
Laboratory Water Quench Plus																	
Temper @ 1000	AAA	88	102	23	80	0.86	19.0	<-120	<-120	<-120	<-120	-100	---	---	120	110	75
Temper @ 1100	AAC	76	88	22	81	0.86	12.0	<-120	<-120	<-120	<-120	<-120	---	---	240	210	150
Temper @ 1200	AAE	73	82	29	83	0.89	11.0	<-120	<-120	<-120	<-120	<-120	---	---	240	240	240

C	Mn	P	S	Si	Cu	Ni	Cr	Mo	V	Cb	Al
0.024	1.40	0.014	0.002	0.75	1.30	1.93	0.006	0.004	0.002	0.02	0.029

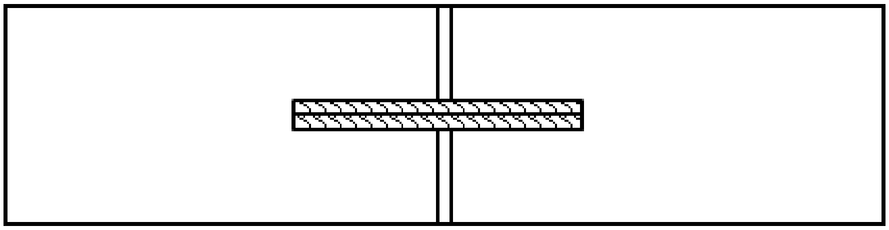
		Tensile Properties					Hard	Charpy V-Notch Transition Temperature					Charpy V-Notch Energy				
		YS	TS	EL	RA	YS/TS	HRc	20 ft-lb	35 ft-lb	60 ft-lb	15 mils	50% FAT	70	0	-40	-80	-120
Laboratory Water Quench Plus																	
Temper @ 1000	BAA	93	106	26	76	0.88	21.0	-120	-110	-100	-110	>-40	---	---	85	70	20
Temper @ 1100	BAC	83	96	29	78	0.86	16.0	<-120	<-120	<-120	<-120	-120	---	---	180	150	130
Temper @ 1200	BAE	79	91	29	80	0.87	14.0	<-120	<-120	<-120	<-120	-120	---	---	240	240	130

C	Mn	P	S	Si	Cu	Ni	Cr	Mo	V	Cb	Al
0.024	1.40	0.014	0.020	0.75	2.04	1.92	0.006	0.004	0.002	0.02	0.027

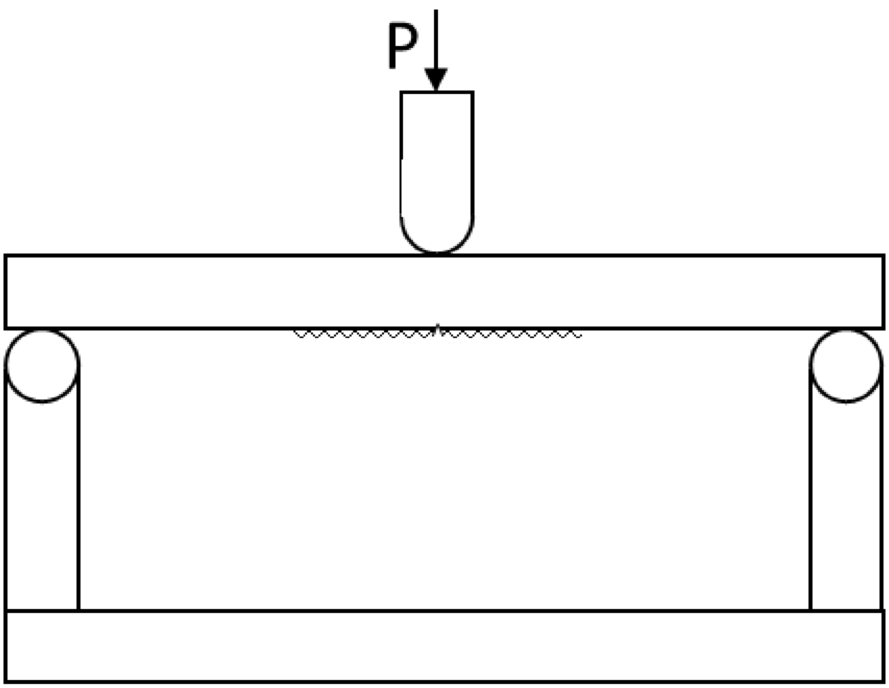
		Tensile Properties					Hard	Charpy V-Notch Transition Temperature					Charpy V-Notch Energy				
		YS	TS	EL	RA	YS/TS	HRc	20 ft-lb	35 ft-lb	60 ft-lb	15 mils	50% FAT	70	0	-40	-80	-120
Laboratory Water Quench Plus																	
Temper @ 1000	DAA	101	112	25	71	0.90	22.0	-110	-60	-40	-100	-30	---	---	65	25	10
Temper @ 1100	DAC	91	99	28	76	0.92	20.0	<-120	-110	-90	<-120	-60	---	---	120	100	40
Temper @ 1200	DAE	82	91	28	78	0.90	17.0	<-120	<-120	<-120	<-120	<-120	---	---	240	160	130

*Average of Duplicate Test Specimens

FIGURE 1



Plan View of Test Specimen Showing Weld-Bead Locations



Loading of Test Specimen

Figure 1. Lehigh Slow-Notch-Bend Test Procedure

APPENDIX I – Corrosion-Test Performance of Developmental Steels

Table A. Results of First Set of Accelerated Corrosion Tests*at 5% NaCI

Last Updated: 13 March 2008		**Exposure Time (Days)**			
		7	**21**	**35**	**70**
Project	**Steel**	**Total Thickness Loss (microns)**			
CCT_L1	9	58	207	361	844
CCT_L1	10	50	185	327	940
CCT_L1	12	39	145	262	322**
CCT_L1	J	54	178	343	642
CCT_L1	K	53	161	287	618
CCT_L1	M	50	167	265	577
CCT_L1	E	53	172	308	561
CCT_L1	F	53	159	283	598
CCT_L1	H	55	153	282	567
CCT_L1	HPS100W (L)	59	211	443	924
CCT_L1	A588 (M)	68	260	463	944
CCT_L1	A36 (1) (0.30%Cu)	69	249	440	925

*For detailed chemical compositions of the above steels, see Table 1. **An anomalous result

The results of the :first accelerated corrosion test on developmental steels exposed at 5% NaCI salt spray are listed in Table A, above. The very first group. Steels 9,10,12 was a fortuitous choice suggested in the Contract Proposal to FHWA because it alerted us to the fact that 4% chromium as in 8tee112 increased the hardenability of the base CuNi precipitation strengthened steel such that the steel hardened even when air-cooled and therefore was embrittled with respect to tensile ductility and fracture toughness. Consequently, as reported, the composition of subsequent steels such as J,K, and M involved elimination of the carbide forming elements (Cr, Mo, and V) and the formulation of a statistical series of four elemental changes, carbon, silicon, copper, and nickel. The corrosion tests reported in Table A began to indicate the direction of the results with respect to the elemental alloy changes. Note that all the developmental steels exhibited less weight loss than the standard A588 steel, which as previously noted is the most widely used bridge steel. However, in applications where deicing salts are used extensively and at seaside applications, it no longer forms an impervious oxide film.

In a March 2010 report, Nippon Steel noted that U.S. Steel's CorTen B steel, now adopted by ASTM as A588, was an excellent inland bridge steel but that when salt solution was involved, major improvements were necessary, which they achieved by eliminating chromium and adding 3% Nickel. This result is consistent with previous ATLSS results that were previously mentioned and which will be reported again. Steels J, K, and M Involve additions of Cu, Ni, or Cu and Ni, The best steel is M, which involves the addition of Cu and Ni to individual totals ·of 2%, and results in one of the best steels among all steels evaluated. Steels E, F, and H each involved the addition of silicon to 2% and an increase in nickel to 2%, Steel F, and Ni and Cu to 2%, Steel H. As shown in Table A the three steels exhibited good good corrosion resistance but were not pursued because the 2% silicon embrittled the steels, both ductility and fracture toughness.

Table B. Results of Second Set of Corrosion Tests at 5% NaCI

	Days	**7**	**21**	**35**	**50**	**70**
CCT_L2	A	52	176	282		618
CCT_L2	B	47	155	266		578
CCT_L2	D	47	147	231		559
CCT_L2	U	54	167	261'		486
CCT_L2	V	51	184	283		552
CCT_L2	W	56	190	299		585
CCT_L2	A852 (L)	55	209	398		848
CCT_L2	A588 (L)	65	246	458		910
CCT_L2	A36 (M) (0.02%Cu)	71	276	564		1078

Provided By (L)=Lehigh
(M)=Mittal

As for the CCT-LI series the six developmental steels in Series CCT-L2, Steels A,B,D,U,V, and W, all exhibited relatively good corrosion resistance compared with the benchmark steels, including A588. The U,V,W steels were not pursued because they each contained 3% silicon and all three steels were severely embrittled. In contrast, the mechanical properties of Steels A, B, and D were outstanding as illustrated in Table B. Taking all factors into account, Steel D was considered to be the best of the 23 developmental steels. The Steel D composition was repeated as a SOD-pound heat to facilitate weldability tests and as reported in that section, the results were outstanding.

Table C.
Results of Third Set of Thickness-Loss Data: Developmental Steels
CCT Exposure Protocol: SAE J2334 (MODt): Solution 5% NaCI

Developmental Steels, 5% NaCl Soln J2334 Mod.						
Project	Steel	Total Thickness Loss (microns)				
		Exposure Time (Days) (1 CCTcycle/day)				
		7	21	35	70	100 (extrap)*
CCT L1	9	58	207	361	844	1210*
CCT L1	10	50	185	327	940	1340*
CCT L1	12	39	145	262	322	700*
CCT L1	J	54	178	343	642	870*
CCT L1	K	53	161	287	618	850*
CCT.-L1	M	50	167	265	577	820*
CCT·L1	E	53	172	308	561	740*
CCT.-L1	F	53	159	283	598	840*
CCT_L1	H	55	153	282	567	770*
		10	20	40	70	100
CCT L2	A	52	176	282	618	885
CCT_L2	B	47	155	266	578	851
CCT L2	D	47	147	231	559	703
CCT L2	U	54	167	261	486	824
CCT L2	V	51	184	283	552	825
CCTJ2	W	56	190	299	585	797
CCT L2	R		248	428	677	1276
CCT L2	S		195	420	667	866
CCT L2	T		151	389	650	1001
CCT L2	X		218	497	867	1124
CCT L2	Y		168	362	666	1202
CCT L2	Z		167	318	557	817

Table C adds Steels R,S,T,X,Y, and Z to the previously reported steels in CCT-L1 & L2 and adds the data for 100 days exposure. As for 70 days exposure, the lowest weight loss after 100 days exposure along with excellent mechanical properties and we1dability is for Steel D. The addition of Steels R,S,T,X,Y, and Z constituted an attempt to reexamine the effect of chromium, inasmuch as it is the primary alloying element in stainless steels. However, at 2% chromium (Steel R) .the weight loss is high, at 4% (Steel S) it is reasonably good but the steel was severely embrittled as was the case for 6% (Steel T). A second attempt at 2%, 4%, and 4% with 2% silicon, Steels X,Y, and Z, respectively, provided a similar picture on the basis that 2%Cr does not significantly improve the corrosion resistance and 4% results in severe embrittlement. Consequently, chromium has been written off as a desirable addition to improve resistance to NaCl exposure unless' accompanied with at least 8%nickel to maintain an austenitic condition.

Based on the preceding results, the five best steels, D as DD, M, S as SS, Y, and R and two benchmark steels, A36 and A588 were selected for further salt-spray chamber testing at 1% and at 3% NaCl. The results which are reported earlier are added here for testing completeness.

Steel	Thickness Loss in microns after Exposure at Indicated NaCl Concentration		
	1%	3%	5%
DD(D) (2%Ni+2%Cu)	430	679	550
M (2%Ni+2%Cu)	436	687	577
SS (4%Cr)	833	737	667
Y (1%Cu+4%Cr)	1000	834	66.6
R (2% Cr)	1238	1199	677
5N (A588)	1037	1250	910
A36	1134	1400	1078

Note that Steel DD, which is identical to Steel D has the best corrosion resistance at all three NaCl concentrations; also that Steel M with the same composition as Steel DD(D) has essentially the same "best" resistance as DD(D). The fact that some significant inconsistencies exist with respect to the relative corrosiveness of NaCI - 1% vs 3% vs 5% would be disconcerting and blamed on poor experimental procedure, except that the order of resistance is the same for all steels tested. Consequently, there is no question about Steel D being the best steel to resist NaCI as shown in the above table. Compared withA588, SteelD is 1037/430=2.41 times better at 1% NaCI, 1250/679=1.84 times better at 3% NaCI and 910/550= 1.65 times better at 5% NaCl: an average of 1.97, essentially twice as good as A588.

The excellent performance of the 2%Cu-2%Ni Steel D justifies the proposal that a commercial 200-ton heat be produced to (1) confirm its excellent mechanical properties and weldablity, (2) conduct large-scale prototype tests, and (3) retain sufficient steel slabs that can be rolled to structural components to fabricate and erect three highway bridges at locations selected by FHWA where improved corrosion resistance is necessary to assure safe long-term operation. This recommendation is further supported by the lifecycle- cost evaluation illustrated in Appendix II.

APPENDIX II – Life-Cycle-Cost Analysis Comparing Steels D and A588

Life-Cycle-Cost Analysis Comparing Steel D with A588

1. Introduction

This appendix presents the details and results of the life-cycle cost (LCC) analysis comparing Steel D with A588 performed by the ATLSS research group (Dr. Dan M. Frangopol and Nader M. Okasha, GRA). It was prepared to comply with the requirements of the FHWA contract and incorporated in its final report. Due to its improved chemical compound, the material cost of Steel D is higher than that of A588. However, the improvement in chemical provides greater resistance to corrosion, which in turn, makes a structure made of this steel capable of lasting for considerably longer periods without the need for local repairs or repainting than a structure made of A588 Steel. Accordingly, the feasibility of Steel D can be gauged fairly only if a LCC analysis is considered.

Therefore, the objective of the task performed by the ATLSS research group is to compare the LCC of a steel bridge component made of Steel D and the LCC of a steel bridge component made of A588 Steel (compositions of steels are available in previously presented Table 1).

2. Input data

For conducting this LCC analysis, three types of input information are required: the geometric details of the structural component considered, the costs of the items involved in the process (material, inspection, repainting, etc), and the frequency of the actions implemented over the life-cycle of the bridge component.

Mr. Ronald Medlock (vice president of technical services, High Steel Structures, Inc.) has provided in a personal communication the geometric details of a typical steel bridge girder. He indicated that a typical steel bridge girder spans about 120 feet, has a depth of 6 feet, web thickness of ½ inch, flange width of 2 feet, and flange thickness of 2, 2½ inches for both top and bottom, with splices at the 1/3 points. Figure A1 shows a cross section of the girder.

Table A1 shows the costs of the various items considered in the analysis along with the references where these costs were obtained from. As shown in the table, the only cost different in the two types of steel compared in this analysis is the material cost. It should be noted that the costs obtained from Toussaint *et al.* (2004) were originally in monetary units of Euros. The Euro units were converted to Dollars using 1 Euro=1.2257 Dollars taken as of 06-01-2010 from Google.

Table A2 shows the frequency of the actions considered to be implemented over the life-cycle of the bridge component. The frequencies associated with the A588 girder are obtained from Toussaint *et al.* (2004). It was determined that the time-interval between consecutive repaintings of the Steel D girder would be twice as long as those of the A588 Steel girder. The same was considered for the local repair frequency. However, it was determined that the inspection frequency is the same in both steel type girders.

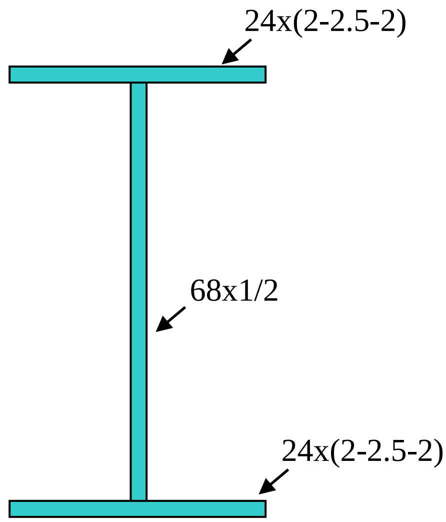


Figure A1. Cross section of bridge girder (dimensions are in inches).

Table A.1. Cost of material and actions considered in the LCC analysis.

Item	Cost		References
	A588 Steel	Steel D	
Total initial cost of girder as fabricated ($/ton)	1400*	1700**	(*Medlock, R. 2010) (**Gross, J.)
Re-painting cost ($/ft^2)	12	12	(Kline, E. 2009)
Inspection cost ($/ft^2)	0.5694	0.5694	(Toussaint *et al.* 2004)
Local repairs cost ($/ft^2)	0.5694	0.5694	(Toussaint *et al.* 2004)

Table A.2. Frequency of actions considered in the LCC analysis.

Action	Frequency (year)	
	A588 Steel	Steel D
Re-painting frequency	30	60
Inspection frequency	10	10
Local repairs frequency	10	20

3. Computation of the initial costs

Cross-sectional area at support = $24\times2\times2+0.5\times(6\times12 - 2\times2) = 130\ \text{in}^2$

Cross-sectional area at midspan = $24\times2.5\times2+0.5\times(6\times12 - 2.5\times2) = 153.5\ \text{in}^2$

Volume= $120\times12\times(130\times2/3+153.5/3) = 198480\ \text{in}^3$

$= 187200\times(0.0254)^3 = 3.2525\ \text{m}^3$

Density = $7.85\ \text{ton/m}^3$

Weight = $7.85 \times 3.2525 = 25.5322$ ton

Area of total surface = $120 \times (2\times6\times12+24\times3 - 2\times0.5)/12 = 2150\ \text{ft}^2$

Initial cost of A588 girder = $1400 \times 25.5322 = \$35745$

Initial cost of D girder = $1700 \times 25.5322 = \$43405$

4. Computation of the LCC

The life-cycle computations are performed as follows:

The LCC cost is equal to the initial cost for the both steel girders. This LCC is constant until the first inspection/maintenance action is applied. Each time an action is applied, its cost is added to the LCC. The costs of application of these actions are:

Inspection cost = 0.5694 × 2150 = $1224.2

Local repair cost = 0.5694 × 2150 = $1224.2

Re-painting cost = 12× 2150 = $25800

An inspection is applied to both steel type girders at the year 10. In addition, a local repair is applied to the A588 girder but not the Steel D girder. Therefore, the LCC becomes:

LCC of A588 girder = 35745 + 1224.2 + 1224.2 = $38193.4

LCC of Steel D girder = 43405 + 1224.2 = $44629.2

This LCC is constant again until the next maintenance action. At the year 20, an additional inspection is applied to both steel type girders. Also, a local repair is applied to both steel type girders. Therefore, the LCC becomes:

LCC of A588 girder = 38193.4 + 1224.2 + 1224.2 = $40641.8

LCC of Steel D girder = 44629.2 + 1224.2 + 1224.2 = $47077.6

At the year 30, an additional inspection is applied again to both steel type girders. Also, a local repair and a repainting are applied to the A588 girder but not the Steel D girder. Therefore, the LCC becomes:

LCC of A588 girder = 40641.8 + 1224.2 + 1224.2 + 25800 = $68890.2

LCC of Steel D girder = 47077.6 + 1224.2 = $48301.8

The LCC keeps accumulating in this manner until the service life, assumed as 100 years in this study, is reached. The final LCC at the year 100 is found as:

LCC of A588 girder = $135179

LCC of Steel D girder = $85118

Figure A2 shows the change in LCC in time for both steel type girders. Also, Figure A3 shows the linear relationship obtained by fitting a straight line between the initial cost and the final cumulative LCC cost for both steels.

Clearly, the final LCC of the A588 girder is about 60% higher than the LCC of the Steel D girder. This is despite the fact that the initial cost of the A588 Steel girder is about 15% lower than the Steel D girder. Therefore, it is conclude that the life-cycle cost analysis shows lower cumulative cost over 100 years for a typical Steel D girder than that of a typical A 588 steel girder.

5. Conclusions

In this appendix, the computations performed and conclusions made by the ATLSS research group with regard to the LCC analysis comparing Steel D with A588 are presented. The motivation behind this project is to determine whether Steel D has lower LCC compared to A588. Therefore, the objective of the task performed by the ATLSS research group is to compare the LCC of a steel bridge component made of Steel D and the LCC of a steel bridge component made of A588 Steel given the different maintenance requirements for each steel type girder.

It is concluded from the results of the LCC analysis that the new Steel D is indeed cost-effective over the long run. The life-cycle cost-effectiveness of the Steel D increases over the service life of the component.

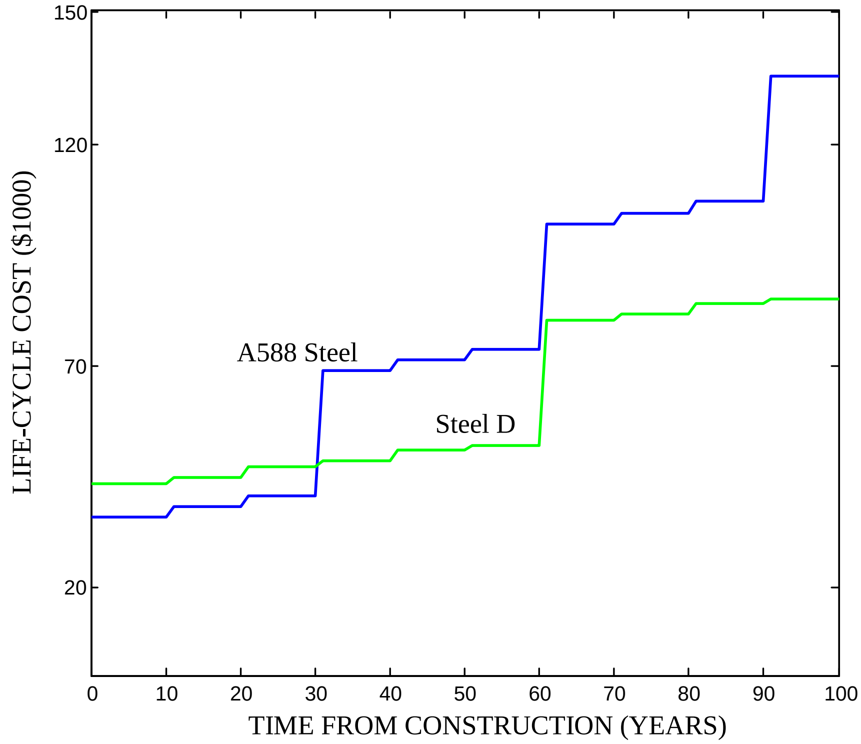


Figure A2. The change of the total LCC with time for a typical steel girder made of A588 Steel and Steel D.

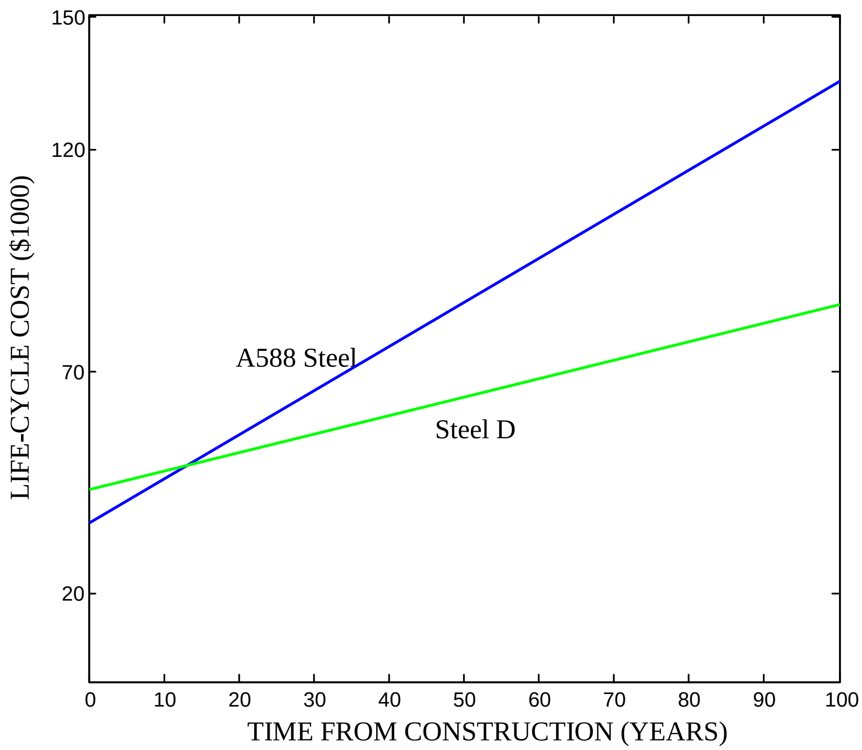


Figure A3. A linear relationship between initial and the total LCC for a typical steel girder made of A588 Steel and Steel D.