4D Journal

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Lake Hodges Stress Ribbon Bridge

by T.Y. Lin International

On May 15, 2009 the world's longest stress ribbon bridge opened in San Diego, California. This bridge was designed by T.Y. Lin International (TYLI) for the San Dieguito River Park. The stress ribbon bridge type, while used with some regularity in Europe, is unique regionally; as to date only five other stress ribbon bridges have been built in North America. At Lake Hodges, a 16 inch thick concrete deck was used to span 330 ft between supports for an amazing depth to span ratio of 1:248. The result is a thin ribbon of concrete with very low visual impact to its natural setting across the lake. The bridge can achieve such a remarkable depth to span ratio since it is actually a cable supported bridge with the bearing cables embedded within its concrete deck. The bridge was post-tensioned to close the transverse joints between the precast deck panels and give it stiffness for live loads. Since the bridge was built by suspending precast panels from bearing cables, no falsework was required. This makes this bridge type ideally suited to water and canyon crossings and sites with difficult access or environmentally sensitive areas.

Although the end result is a simple ribbon of concrete that follows a cantenary shape between supports, the design required complex analytical methods to capture the non-linear behavior of the cable system and the time-dependent effects from concrete creep and shrinkage. The design also required stage construction analysis to capture the stresses that are locked-in as the bridge was constructed.

The LARSA 4D finite element program was used since it has the ability to analyze all of these complexities. Cable elements were used to model the bearing cables and post-tensioning tendons while beam elements were used to model the concrete panels and cast-in-place regions. Each section of the bridge had three discrete members, bearing cable, post-tensioning tendon and concrete beam, which were constrained together at the nodes.

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What's Happening?

Latest News from LARSA, Inc.

by Joshua Tauberer

Director of Software Architecture

We have been hard at work as always improving our software and responding to our clients' needs. Here are some recent developments.

Documentation

Six new manuals have been finished. Three are training manuals, helping you get up to speed on modeling basic to advanced bridges with LARSA 4D. Topics covered include 3D straight and curved bridges, post-tensioning, nonprismatic variation, Staged Construction Analysis, and live load using influence lines and surfaces. They are:

- LARSA 4D Introductory Training Manual
- LARSA 4D Basic Training Manual for Bridge Projects
- LARSA 4D Advanced Training Manual for Bridge Projects

There are also new manuals for LARSA Section Composer, which doubles as a tutorial, for the LARSA 4D Steel Plate Girder Design Module for AASHTO LRFD, and for LARSA 4D's Cable Tension Optimization Tools.

You can find the manuals on our website, www.LARSA4D.com. Click Support > Client Support Center > Product Documentation.

Website

You never get a second chance to make a first impression, so we took a hard look at our website www.LARSA4D.com and decided it was time for a refresh. Besides making it more surfer-friendly, you can find on the website our interactive project portfolio, back issues of the 4D Journal, and our brochures.

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Visit Us

We'll be at the following upcoming conferences:

PCI/FIB Annual Convention and Bridge Conference
May 29 - June 2, 2010
Gaylord National Resort
Washington, D.C.

International Bridge Conference
June 6-9, 2010
David L. Lawrence Convention Center
Pittsburgh, PA

US National & Canadian Conference on Earthquake Engineering
July 25-29, 2010
Westin Harbor Castle Hotel
Toronto, Canada

ASBI Conference 2010
October 11-12, 2010
Vancouver, Canada

Up Next

In the next issue of LARSA's 4D Journal:

Arbour Stone Bridge

A 120 meter twin steel arch pedestrian bridge built in The City of Calgary, Alberta, Canada. The consulting engineers of Infinity Group, Ltd. of North Vancouver talk about the project and LARSA 4D's role in its creation.

Front Cover: Lake Hodges Stress Ribbon Bridge, San Diego, California courtesy of T.Y.Lin International

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LARSA 4D is analysis and design software for bridges, buildings, and other structures, developed by LARSA, Inc. in New York, USA.

This journal is distributed as a courtesy to our clients and corporate friends. We welcome feedback and suggestions for future stories to info@LARSA4D.com.

Features On Demand

Our team is well known for providing timely and useful technical support, webinars, and on-site training. Our newest support system is called “Features On Demand.” This allows us to provide new features directly to waiting clients without having to go through our normal software release cycle and without the user having to uninstall and reinstall an updated version of LARSA 4D (which can be inconvenient in corporate offices). New features that we have been able to provide within hours include automatic generation of bridge path coordinate systems based on existing model geometry and computing the length of patch loading in influence line analysis.

As the lean, mean, structure-solving machine, we always are on the look-out for new ways to deliver on our promise for unparalleled support services. “LARSA Live” is another new tool in our arsenal for rapidly providing new custom-build tools to our clients. LARSA Live allows users to run LARSA 4D off the Internet without having to install it. This means users can run an old and new version of LARSA 4D at the same time, and the new version can be tested without requiring IT’s help to install the program.

We’re Going Global

The software will soon be available in a select few languages besides English. Our British clients have told us they can’t understand our software’s New York drawl.

The Shell Element

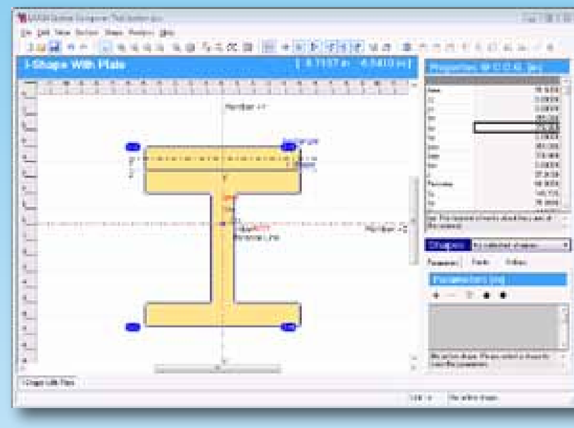
It’s a common misconception that Indiana Jones was searching for the Holy Grail. In fact, he was looking for the Unified Shell Element, an advanced element formulation that will allow users to combine separate plate and membrane models into a single shell element.

After adding a “thick plate” element formulation, we added drilling degrees of freedom to “membrane element” formulation. Drilling degrees of freedom provide for more realistic modeling when the shell is subject to a torque about its normal axis.

We have also added shell element end offsets, much like member end offsets, which is useful for creating rigid connections between a girder and a deck. Twelve end offset values (x, y, z for each node) can be entered for each plate.

Cable Reincarnation

What does reincarnation have to do with nonlinear analysis? The cable element’s “rebirth” capability is when it goes in and out of an analysis depending on whether it is under tension. When the cable goes into compression, it is taken out of the analysis because the cable has no compressive strength. This can make nonlinear convergence more difficult, so we

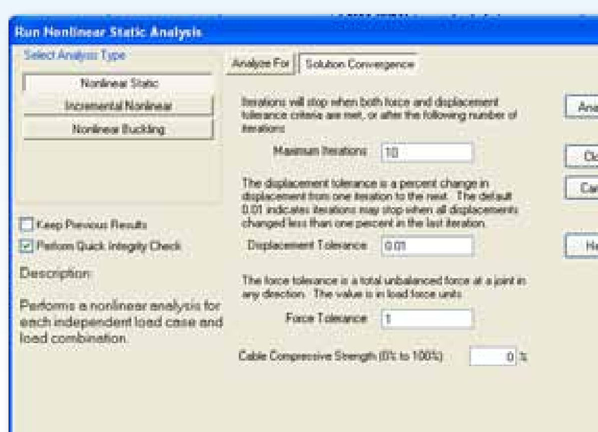


LARSA 4D and Section Composer are being updated for composite construction.

are adding a new option: cable compressive strength. When this option is used, cables in compression will retain a small fraction of their tensile strength.

Composite Sections (Almost!)

In software development it is notoriously difficult to estimate how long it will take to deliver on a promise. More than three years ago we began work on composite section construction. This major update to the LARSA 4D analysis engine would allow member cross-sections to be built up over time in the Staged Construction Analysis. A girder might be placed first with a deck (modeled in the cross-section) cast later on. Though the parts are combined into a single line element, they have different time-dependent effects such as creep. This update also includes nonlinear thermal gradients. We began previewing this update on a limited basis in 2010 and expect a public preview later this year. •



Cable Compressive Strength is a new option for nonlinear analysis types to make convergence easier.

Lake Hodges-Stress Ribbon Bridge

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Restrained nodes were sufficient to model the fixed north abutment. However, the four pile shafts at the south abutment were individually discretized in the model to account for their non-linear behaviour of the soil.

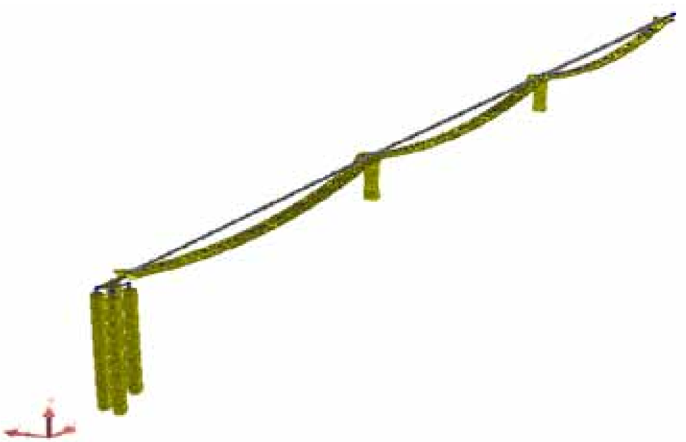

Fig. 1 – Rendering of LARSA 4D Model

Architectural and Structural Solution

TYLI studied several bridge types during the type selection process. Other structure types that were considered included: reinforced concrete slab, precast concrete girders, cast-in-place prestressed concrete box girder, timber glue-lam girders, prefabricated steel trusses as well as suspension and cable stayed alternatives. The TYLI led team and the River Park worked together to select a bridge type that would be economically feasible to construct and easy to maintain, would compliment the site and most importantly would be compatible with the River Park's mission:

- To preserve and restore land within the Focused Planning Area of the San Dieguito River Park as a regional open space greenway and park system that protects the natural waterways and the natural and cultural resources and sensitive lands and provides compatible recreational opportunities, including water related uses, that do not damage sensitive lands.
- To provide a continuous and coordinated system of preserved lands with a connecting corridor of walking, equestrian, and bicycle trails, encompassing the San Dieguito River Valley from the ocean to the river's source.

In the end, the stress ribbon bridge type prevailed for several reasons. Firstly, this bridge type could be built with the 330 ft spans required to limit the number of permanent supports within the lake to two, and this could be accomplished with minimal visual impact to the natural habitat since the deck would be only 16 inches thick. Secondly, since the superstructure would be built of precast panels suspended from bearing cables, there would be no need for falsework, which would further reduce impacts to the environmentally sensitive habitat and also eliminate the expense of constructing falsework over a major waterway.

Thirdly, constructed of a continuous ribbon of prestressed concrete, using common materials and construction methods, the bridge would be economical to construct, extremely durable and essentially maintenance free. And finally, the stress ribbon design would result in a beautiful and unique pedestrian bridge that would be a landmark for the San Dieguito River Park and the region.

The firm Safdie Rabines Architects (SRA) was brought on board to help with aesthetic details. SRA was instrumental in the architectural shaping of the bridge. Jiri Strasky, who is well known as a pioneer of the stress ribbon bridge type, was hired to provide input on the conceptual design and to perform the independent check of the bridge.

Aesthetics played a major role in the River Park's decision to go with the stress ribbon bridge type. Ricardo Rabines of SRA developed some of the early sketches of the bridge and felt that aesthetically the stress ribbon bridge was ideal for this setting across Lake Hodges, since it blends harmoniously into the surroundings in both wet and dry conditions. His description for the stress ribbon bridge is one of "floating" over the water, or "nesting" above the dry lakebed. See Fig. 2 and 3.

Stage Construction Model

A stage construction analysis was conducted where the bridge was "constructed" step-by-step within the computer model. First the substructure elements were constructed, then the bearing cables were installed, the weight from the precast panels and cast-in-place concrete was applied, the concrete stiffness was activated, then the post tensioning force was applied and so forth. This stage construction analysis was necessary to capture the stresses that were locked into each component - bearing cable, concrete ribbon, and post-tensioning tendon - as the bridge was constructed. The stage construction analysis also allowed the time dependent effects of creep and shrinkage to be analyzed.

Time Dependent Effects

The creep and shrinkage functions from the CEB-FIP 78 code built into the LARSA 4D program were used. The casting day of each member and other creep and shrinkage parameters were input into the program. After the bridge was constructed in the model, the design load combinations were applied. Next, 50 years of creep and shrinkage were applied in steps and the load combinations were analyzed again. In the end the load combination that included dead load, added dead load, live load and a drop in temperature of 35 degrees Fahrenheit, all applied after 50 years of creep and shrinkage, controlled the design.

Thermal Loading Combination

The behavior of the continuous concrete ribbon was such that each load in the controlling load combination tended to pull on the ribbon and when combined resulted in tension in the concrete. Initially the post tensioning put the concrete ribbon into compression, but this compression was reduced by the added dead load. Creep and shrinkage had the effect of causing the bridge to shorten, move upward, and further reduced the amount of pre-compression. The applied live load and a drop in temperature of 35 degrees Fahrenheit further acted to put the concrete ribbon into tension. The design intent was to keep the concrete ribbon from going into tension under permanent loads and to keep tension to a reasonable level under the extreme case of full live load and thermal loading.

Fig. 2 – Rendering of completed bridge, looking west, lake filled with water

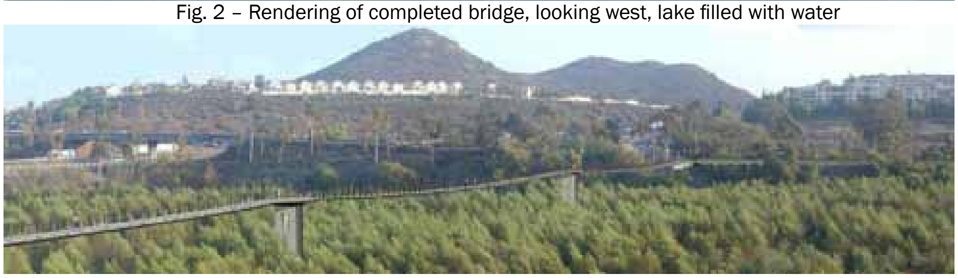

Fig. 3 – Rendering of completed bridge, looking southeast, dry lake condition

The controlling load combination for the superstructure was as follows: 1.0 D + 1.0 L + 1.0 PS + 1.0 T + 1.0 C&S

Where D is dead load, L is live load, PS is prestress, T is thermal loading and C&S is creep and shrinkage. The allowable stress in this condition was 125% of the basic allowable stress. Allowable extreme fiber tension stress in the concrete under this combination was:

$$f_t = 1.25 \, x \, 3\sqrt{f'c} \text{ [psi]}$$

For the superstructure concrete, which has a specified nominal compression strength of $f'c$ = 6,000 psi, the allowable tension stress is 290 psi.

Since the probability of the bridge being simultaneously subjected to maximum live load and maximum thermal loading is very low, a reduction of 25% to one or the other was allowed. The result was that a post-tensioning force of 4,600 kips was required to meet the above design criterion.

Capacity for Overload

One of the interesting characteristics of the stress ribbon bridge type is its remarkable capacity for overload. LARSA 4D was used to investigate this. In this analytical study, the pedestrian live loading was incrementally increased. At a four-fold increase in the live loading, the study showed the bridge could easily carry the increased load with only a nominal stress increase to the bearing and post-tensioning cables. The extra load carrying capacity instead comes from the change in the cable geometry. Increased cable sag results in the corresponding increase in the vertical load carrying component of the cables.

Dynamic Analysis

A dynamic analysis was required to assess the performance of the bridge to live load vibrations, heavy winds, and seismic actions.

Since the stiffness of the bridge changes with applied loading, a "stressed Eigen-value" analysis was required to determine the natural frequencies and mode shapes. This is a special type of Eigen-value analysis in which the stiffness matrix includes the static loads and the deformed geometry of the structure. Within the stage construction model, the bridge was constructed, post-tensioning was applied, the superimposed dead load was applied, then the Eigen-values were determined. It was important to determined the Eigen-values at this state of stress in the bridge to get the correct stiffness. The first three mode shapes are shown in Fig. 4.

The results of the analysis showed that modes 1 and 2 each have less than 0.02 percent mass participation, while mode 3 captures about 20 percent mass participation. Thus, mode 3 is the vertical mode used to investigate the bridge's sensitivity to excitation from pedestrians.

Live Load Vibrations

The vibrations from pedestrian loading on the bridge were evaluated based on simplified approaches from the British and Ontario bridge codes. Both codes specify a simple design procedure that determines the vertical acceleration resulting from the passage of one pedestrian walking with a pace equal to the fundamental natural frequency of the bridge. The procedure is for a bridge excitation by one pedestrian and no allowance is made for multiple random arrivals of pedestrians. For footbridges up to three spans, the estimated vertical acceleration is:

$$a = 4\pi^2 f_1^2 \, yK\psi \text{ [ft/s}^2\text{]}$$

Where f_1 = fundamental natural frequency of the bridge [Hz]
y = static deflection at mid-span for a force of 160 lbs [ft]
K = configuration factor
Ψ = dynamic response factor

The fundamental frequency and static deflection at mid-span were determined from the finite element model. The stressed Eigen-value analysis resulted in fundamental vertical frequency of f_1 = 0.74 Hz. The static deflection at mid-span for a 160 lb force (static weight of one pedestrian) was y = 0.0014 ft.

The configuration factor, K, is based on the number of spans and is 1.0 for a single span, 0.7 for a two-span, and between

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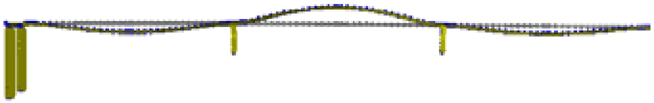
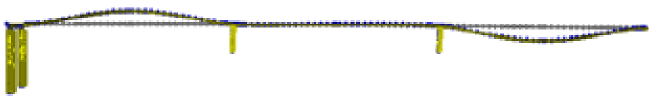
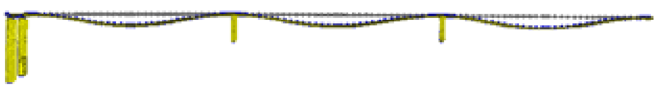


a) Mode 1 – 0.43 Hz



b) Mode 2 – 0.45 Hz



c) Mode 3 – 0.74 Hz

Fig. 4 – Mode Shapes from the Stressed Eigen-value Analysis

0.6 and 0.9 for a three-span bridge. For the three-span Lake Hodges Bridge, the upper limit of K= 0.9 was conservatively used.

The dynamic response factor, Ψ, is given in graphical form as a function of the span length and damping ratio. For the Lake Hodges bridge, a damping ratio of 0.010 was used, which is considered to be appropriate for a prestressed concrete footbridge. Using this damping ratio with a 330 ft span resulted in a dynamic response factor of Ψ = 39. Using these values in Equation 4 gives a vertical acceleration of a = 0.33 ft/s^2.

This vertical acceleration must be compared to the allowable limits specified by the codes. For fundamental natural frequencies, f_1 less than 5 Hz, the British code gives a vibrational acceleration serviceability limit of:

$$a_{max} = 0.8 f_1^{0.5} \quad [\text{ft/s}^2]$$

The Ontario code gives a more conservative serviceability limit of:

$$a_{max} = 0.8 f_1^{0.78} \quad [\text{ft/s}^2]$$

For f1 = 0.74 Hz, the allowable limits are 1.4 ft/s^2 and 0.65 ft/s^2, respectively for the British and Ontario codes. The calculated acceleration for the Lake Hodges Bridge, at 0.33 ft/s^2, is well below these allowable limits.

Furthermore, the fundamental vertical frequency of the Lake Hodges Bridge at 0.74 Hz is well outside of the natural frequencies of footfall, which are typically in the range of 1.65 to 2.35 Hz for walking pedestrians or up to 3.5 Hz for running pedestrians. Thus, the analysis determined that live load vibrations would not be a problem for the Lake Hodges Bridge.

Wind Analysis

To verify that the bridge would be stable under heavy winds, a special wind analysis was performed by West Wind Labs to determine the bridge buffeting response. To do this, the aerodynamic load characteristics of the bridge, a description of the aerodynamic turbulence, and the mechanical dynamic properties of the bridge were required. Wind tunnel tests on a 1/10 scale model of the bridge section were performed to determine the aerodynamic load characteristics on the bridge deck, Fig. 5.

Fig. 5 – Wind tunnel testing of a 1/10 scale model

The aerodynamic turbulence field was described by a series of horizontal and vertical wind speed time histories at 30 nodes along the length of the bridge, Fig. 6.

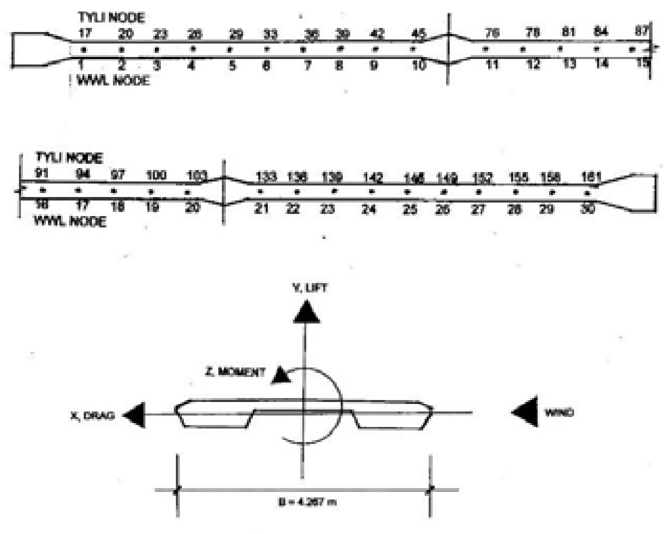


Fig. 6 – Numerical model of the bridge

The dynamic properties, dominant mode shapes and frequencies of the bridge, were determined from the LARSA 4D finite element model.

A numerical simulation procedure was then used. Wind speed time histories at 30 nodes along the bridge deck and 20 modes of vibration were included in the analysis. Numerical simulations for a duration of 5 minutes, with a typical step size of 0.02 seconds, were generated. Bridge stability was evaluated by comparing the modal responses at the end of the simulation to the corresponding modal responses at the beginning of the simulations. Statistics of the steady state modal responses were used to generate a buffeting response. For horizontal winds from the east and west (perpendicular to the axis of the bridge), a numerical simulation was performed for mean wind speeds (here assumed to be a ten minute averaged wind speed) of 67, 71, 76, 80, 85, 89 and 94 mph.

The numerical simulations were performed in smooth flow. At the beginning of each simulation, all modes of vibration began with a modal displacement of unity. Each mode was released in the specified wind, and all were allowed to vibrate freely and simultaneously, allowing for any cross coupling should there be an aerodynamic tendency to do so. Dynamic flutter instabilities (single-degree-of-freedom and coupled multi-degree-of-freedom flutter instabilities) were then identified by the ratios of the modal standard deviations at the end of the simulations to the corresponding initial modal standard deviations. If a modal ratio was greater than unity, then that mode was diverging and the bridge was dynamically unstable.

The results showed that the bridge would remain stable up to a wind speed of 86 mph, which exceeded the maximum expected wind speed at the site.

Seismic Analysis

To evaluate the seismic demands on the bridge, a response spectrum analysis was performed using the mode shapes, which were derived from the stressed Eigen-value analysis. The seismic loading for the site was controlled by a moment magnitude 7.0 event on the Rose Canyon fault, located approximately 14 miles to the southwest. The 1996 California Seismic Hazard Map was used to estimate a peak bedrock acceleration of 0.3g for the site corresponding to the maximum credible earthquake. The response spectrum analysis showed very small seismic demands. At the tops of the piers, transverse demands were less than 1.0 inch, and longitudinal demands were less than 0.6 inches. Displacement demands of this magnitude indicated that the bridge will remain elastic under seismic loading. The robust seismic performance of this bridge is intuitive as the mass of the thin superstructure is small and lateral movement is restrained by a very large cable force. Thus, seismic demands did not control any aspect of the design, which is highly unusual for a bridge in California.

Partial Prestress Design

Although the majority of the superstructure was designed as fully prestressed concrete, using gross section properties to calculate the stresses within the 16 inch thick panels, near the supports this was not feasible due to excessive positive moment demands. The high moment demands are a result of the stress ribbon construction method. When a stress ribbon bridge is post-tensioned, the ends of each span rotate as the concrete ribbon is lifted upward by the prestress force. This results in positive moment demands at the ends of each span that are an order of magnitude greater than the negative moment demands along the rest of the structure. Fig. 7 shows the bending moment demands after post-tensioning and permanent loads have been applied. The negative moment demands are relatively constant with a maximum of -228 kip-ft, The positive moment demands spike near the supports and reach a maximum of 3,370 kip-ft which is 15 times the negative moment demand.

To deal with these large positive bending moments, a partial prestress design method was used. Partial prestress theory accounts for cracking of the prestressed concrete section under service loads. The design methodology is similar to

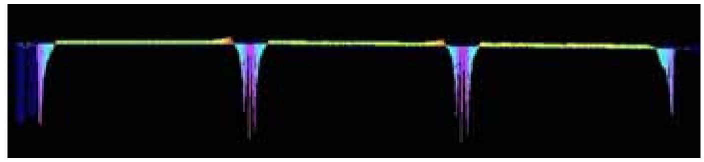


Fig. 7 – Bending moment under permanent loading, Mneg = -228 kip-ft, Mpos = 3,370 kip-ft

working stress design for conventionally reinforced concrete sections. However, the effect of prestressing must be accounted for. The concrete section is allowed to crack and the tensile stresses are resisted by the mild-steel reinforcing bars. Bending moment demands were calculated from the finite element model and a special MathCAD program was written to calculate the resulting stresses in the concrete and steel for each section.

Under the temporary condition before prestress losses, the allowable extreme fiber compression stress in the concrete is limited to $0.55f'_{ci}$. After prestress losses, the allowable stresses in the concrete are $0.40f'_c$ under permanent loads and $0.60f'_c$ under permanent plus live loads. The allowable stress in the mild steel is $0.40f_y$. The allowable stress in the prestressing steel is $0.72f'_s$. Allowable stresses are increased by 25% when temperature or wind is used in the service load combination.

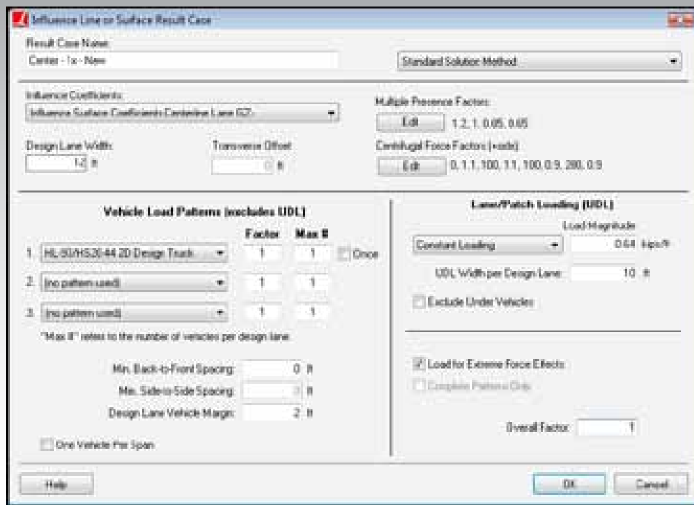
The solution for the Lake Hodges Bridge was to haunch the ends of each span over a 20 ft length from a typical thickness of 16 inches to 36 inches at the face of support. The mild-steel (Grade 60 ksi) reinforcing required at the 36 inch thick section at the face of support consisted of 28 - #10 bars along the bottom edge for a tension steel area of 35.6 in^2 and a reinforcing ratio of 0.60%. The 16 inch thick section 20 ft from face of support required 17 - #10 and 12 - #6 bars for a tension steel area of 26.5 in^2 and a reinforcing ratio of 1.2%.

A unique structure type was used for the 302-meter-long Lake Hodges Bicycle/Pedestrian Bridge. In North America, the stress ribbon bridge type has only been used a handful of times, and world-wide an example of this length has never before been constructed. This special stress ribbon design required specialized analysis procedures, which are typically not necessary for standard bridges. The analyses showed that the Lake Hodges Stress Ribbon Bridge will be safe for dead and live loads and robust for overload, will perform adequately under long-term creep and shrinkage loading, will not be susceptible to live load vibrations, will be stable under the maximum wind loading expected at the site, and will resist loads imposed by the maximum credible earthquake. Considerable effort went into the analysis and design of this bridge. However, the extra effort has resulted in a world-class bridge that will compliment its natural setting across Lake Hodges and within the San Dieguito River Open Space Park and will be a major asset to the people of San Diego County.•

Live Load Analysis with Influence Surfaces

by Joshua Tauberer
Director of Software Architecture

Live load codes impose some of the most computationally intensive requirements on structural analysis. On a bridge deck of a certain size, what are the locations of the traffic lanes and the vehicles on the lanes that produce the greatest moment somewhere on the structure? Design codes add on top of the variability in traffic lane and vehicle locations the additional variability of vehicle type and length. One traffic lane may be loaded with a special permit truck and the remainder with AASHTO LRFD HL-93s, each with variable rear axle spacing.



Influence Surface Results Case Window

Compared to the static analysis which in finite element analysis has the closed-form solution $D=K^{-1}P$ (the inverse of the stiffness matrix multiplied by the load vector), live load analysis is a much more intensive combinatorial problem.

LARSA 4D includes influence analysis using both influence lines and influence surfaces. Influence lines are used in models in which the deck surface is modeled as a part of the girder line-element cross-sections. Each influence line represents a single traffic lane at a user-identified transverse location. The lines may be put together into factored combinations using LARSA 4D's linear result combinations and extreme effect groups tools. Options include vehicle type (standard or user-defined), variable axle spacing, variable vehicle spacing for multiple vehicles, patch loading for UDL, and (in LARSA 4D version 7.5) factors for centrifugal force effects for curved bridges. Also supported is the span-by-span loading requirement of AASHTO LFD. (A tutorial for influence lines can be found in the LARSA 4D Introductory Training Manual for Bridge Projects.)

Influence surfaces are a much more powerful and robust method for live load analysis. With an influence surface computed for the entire deck surface area, LARSA 4D can automatically find the transverse locations of as many traffic lanes as will fit on the surface. This saves the user the time of choosing lane locations, modeling separate influence lines, and forming factored combinations. With an influence surface the separate lane effects are combined using supplied multiple presence factors automatically. Additional options for influence surfaces include traffic lane width, vehicle margins, and UDL width within each traffic lane, and having up to one lane with a different vehicle from the rest for local permit loading. Influence lines and surfaces can be used with curved and skewed bridges. (A tutorial for influence surfaces can be found in the LARSA 4D Advanced Training Manual for Bridge Projects.)

LARSA 4D has introduced new tools for live load analysis in each major update. Starting in version 7.5 of LARSA 4D, influence analysis has been upgraded to be significantly faster especially for wide decks that fit three or more traffic lanes. The analysis also makes use of multi-core/multi-CPU computers, spreading the combinatorial investigation across all available processors. We have found a 2x speed improvement on four processor cores and a 4x speed improvement on eight processor cores, on top of the speed improvements compared to influence surface analysis in version 7.01 found even on single-core computers. The new version also adds support for the span-by-span requirement of AASHTO LFD to influence surfaces with automatic lane positioning.

As with all development at LARSA, new capabilities are always driven by feedback from our clients. We look forward to hearing how we can continue to evolve LARSA 4D's live load analysis to meet all of our clients' project requirements.

What's New To Influence Surface Analysis in LARSA 4D Version 7.5

- Centrifugal force factors can be specified which automatically adjusts vehicle wheel magnitudes. Multiple factors can be specified to apply difference factors on different segments of a single influence surface.
- Some codes require at most one lane of one vehicle type, with the remaining lanes loaded with HL-93. This is now supported.
- AASHTO LFD point loading is now supported for influence surfaces with multiple design lanes.
- Influence analysis is now multi-threaded on computers with multiple processors or multiple cores.
- Influence surface coefficient grid spacing can be larger without as much loss of accuracy because of better load positioning.
- Results are faster and more accurate when more than two vehicles are placed on the surface because of an improved vehicle position search procedure.
- Influence surface design lanes are now able to include UDL without vehicle loading to allow more flexibility in creating load combinations. •