CORRELATING BRIDGE DESIGN PRACTICE WITH OVERLOAD PERMIT POLICY

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Bridge engineers in many states have been designing bridges using working stress methods and AASHTO HS20 live load vehicles for the past 35 years. Recently, the AASHTO Specifications have permitted the use of load factor design for common structure types. During this same period there has been a significant increase in the number of vehicles that greatly exceed legal loads operating with special permits, as well as a marked increase in the weight of legal vehicles. To accommodate these loads, higher stress levels, defined as "operating stresses" in the AASHTO Manual of Maintenance Inspection of Bridges, are allowed at the discretion of the responsible agency. The practice of designing new structures by one set of rules and computing overload capacities by another results in a peculiar situation. The overload capacity of new structures varies widely depending upon construction materials and span length. It is inefficient to have the load capacity of a route segment limited by one or two structures while others have far greater capacity than can possibly be utilized. The purpose of this paper is twofold. First, to alert bridge engineers that they will reduce the usability of their highways by adopting load factor design without a corresponding increase in design live loads. Second, to relate how the California Department of Transportation assures uniform overload capacity at "operating stress" levels in new structures by routinely including a family of standard permit vehicles as one of the loading conditions in a load factor design method.

This paper will cause state bridge engineers to question the adequacy of their present design procedures. It will particularly throw doubt on the prudence of following the Load Factor Design (LFD) method prescribed in the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges using HS20 loads only.

We hope to cause the bridge engineer to consider some serious questions:

- Are the bridges under construction in his state adequate to carry the extra-legal loads that are currently operating there under permits?
- Will they be adequate to accommodate both legal and extra-legal loads anticipated 10 or 15 years from now?
- Is the capacity for overload in his bridges fairly uniform regardless of structure type and span length or is the usability of a particular route segment controlled by a single structure, or structure type?
- Will he make matters better or worse by adopting LFD?

These questions are timely because the nation is about to embark on an accelerated federally funded bridge replacement program. Since this provides an opportunity to upgrade a significant number of structures, it is reasonable that the bridge engineer give serious thought to the load capacity required in these replacement structures.

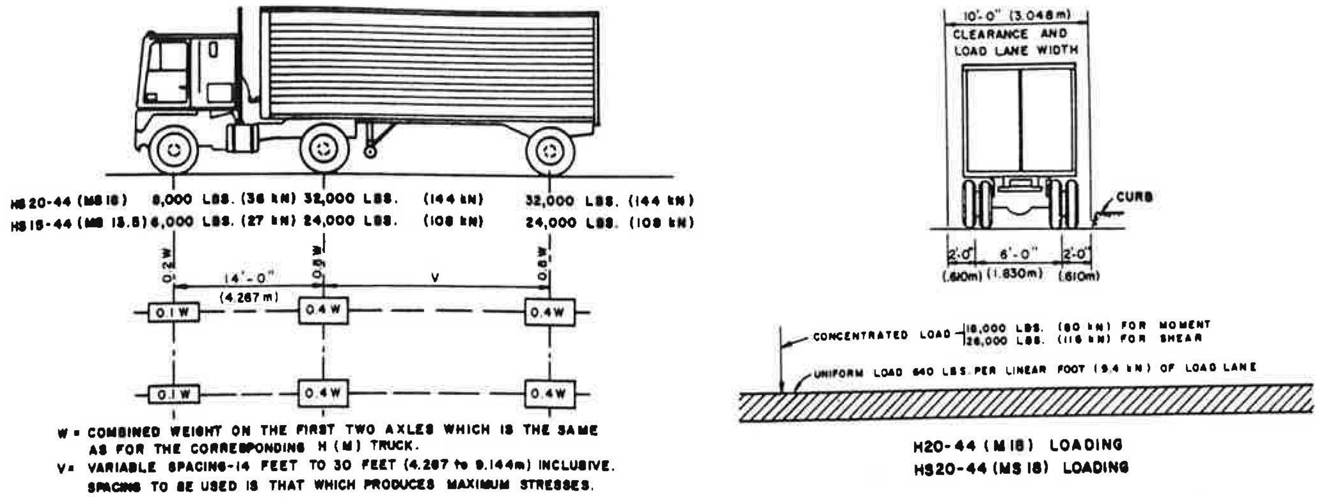
There is another reason for reviewing design loads for states that have adopted, or are thinking of adopting, LFD as standard office procedure. This design method produces structures with lower initial cost than those designed by the Working Stress Design (WSD) method, but the capacity for overload, although more uniform, is lower. Building these lighter structures may be a poor investment.

We, in California, asked ourselves the above questions a few years ago before adopting LFD. We found little correlation between loads and stresses assumed in design and those that occur in operating the structure after it is built. We found a wide variation in overload capacity from bridge to bridge. Most importantly, we found that if we adopted LFD using only an HS20 design load, our newest structures would not be able to carry permit vehicles that had become fairly common on our highways.

Definition of Loads

This paper will be more readily understood if agreement is reached on loading terminology.

The H and HS hypothetical loads are defined in the AASHTO Standard Specifications for Highway Bridges and are illustrated in Figure 1. The HS20 load is routinely used by all but a few states in the design of new bridges on major routes.



HS LOADINGS

Figure 1

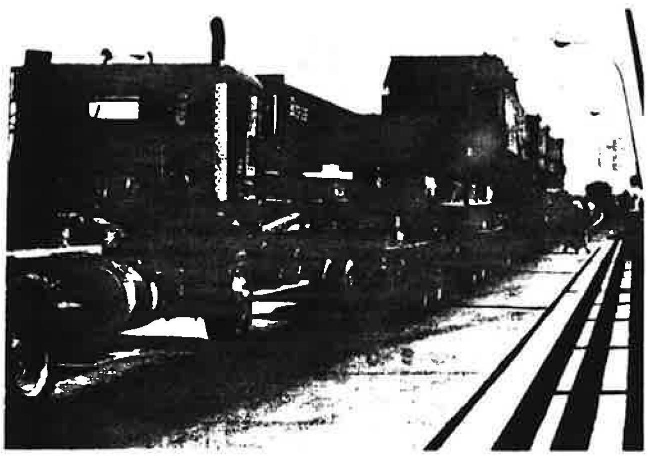


Figure 2

Legal loads are those maximum weights and dimensions of motor vehicles that can operate on highways without special approval from authorities. Federal Aid Amendments of 1974 increased the permissible weight of vehicles operating on the Interstate Highway System to the following levels:

Single Axle Wt	9,060 kg (20,000 pounds)
Tandem Axle Wt	15,400 kg (34,000 pounds)
Gross Wt	36,240 kg (80,000 pounds)

Legal loads vary somewhat from state to state in total weight, weight control on internal axle spacings, and overall truck dimensions. However, in most states they are quite similar to those shown above, which are allowed by the Federal Highway Administration on the Interstate Highway System.

Permit loads are those loads that exceed legal limits (extra-legal) but are allowed to operate on the highway under a permit issued by a regulatory agency. There is a wide variation in extra-legal load permit policy from state to state and the details of these policies are not accurately documented at this time. A National Cooperative Highway Research Program Synthesis, now in progress, entitled "Motor Vehicle Size and Weight Regulation, Enforcement, and Permit Operations", will soon provide information on current practice.

In California maximum axle weights allowed under routine permits are about 50% greater than legal loads. We have devised a family of overload vehicles called "P-loads" which are used in rating structures. (See Figure 6) A cursory review of information prepared commercially for the trucking industry by Heavy-Specialized Carriers Conference indicates that California's permit loads rank with the heaviest in the country.

Permit Policy

California's existing highway system is typical in that it includes bridges of many vintages in diverse states of repair. The structures were designed by various working stress design rules to carry several different AASHTO loadings. Since about 1941 we have used AASHTO HS20, but earlier structures were designed for H15 or even lighter loads.

In addition to legal loads, it has long been California's policy to allow vehicles that exceed legal weight limits to use its highway with controlled permits. To simplify the administration of the permit program, all of the existing structures on the state highway system are being rated using axle configurations and weights that resemble actual permit vehicles. This rating process involves a detailed structural analysis of every bridge.

In each highway district, a permit engineer has data, organized by route and post mile, that relates the live load capacity of each bridge to standardized loads. Using these data and other information concerning limitations on truck dimensions, axle spacings, and truck suspension systems, he can issue permits for extra-legal loads routinely, i.e., without further stress analysis of structures on the route. Last year approximately 200,000 trips were made in California using routine permits. These included loads of up to 110,500 kg (244,000 pounds) carried on vehicles with nine axles.

Besides these routine permits, special permits for even heavier loads are issued. This kind of permit requires a detailed analysis of each structure on the planned route for the specific load and vehicle under consideration. Frequently, certain operating conditions such as driving at reduced speed and limiting other traffic on critical structures, are imposed to reduce stress levels. A load of this type is shown in Figure 2.

Operating Stresses

To fully utilize the mix of existing bridges for permit loads, stresses much higher than design working stresses may be used to determine maximum load capacities. AASHTO has sanctioned the use of these higher stress levels and has defined them as "allowable operating stresses" in the Manual for Maintenance and Inspection of Bridges. A recent review of federal inventory data showed that 27 states use the maximum operating stress levels in rating existing structures, i.e., determining safe load capacities. Other states rate their bridges at lower stress levels, ranging from design working stresses to the maximum operating stress levels.

The rationale for permitting higher stresses for permit loads is based partly on the fact that these loadings are known, whereas during design, future increases in loads must be anticipated. Also, heavy permit loads are thought to be applied infrequently so that fatigue damage is not a major concern.

The relationship between design stresses and operating stresses can be shown by considering the tension flange of a girder constructed of ASTM A-36 steel. For this material, the allowable design working stress is 0.55 F_y or 138,000 kN/m^2 (20,000 psi). The maximum allowable operating stress on this same material is 0.75 F_y or 186,000 kN/m^2 (27,000 psi).

For a concrete structure the spread in reinforcing steel stresses is even greater. For ASTM A-615, Grade 60 bars, the allowable working stress is 230,400 kN/m^2 (24,000 psi) and the operating stress is 345,600 kN/m^2 (36,000 psi).

The practice of designing a structure using one stress level, then rating it for overloads using a higher stress level results in a wide array of overload capacities for newly designed structures. This happens because the reserve capacity in a structure proportioned for one stress level and then reviewed at a higher stress level is derived from lowering the factor of safety for both dead and live loads. Therefore the additional capacity that can be made available for increased live load depends on the ratio of dead load to live load. Typically, for bridges designed by WSD and then reexamined at operating stress levels, concrete structures have a much greater capacity than steel structures.

AASHTO also allows rating of bridges with a load factor method. This method, which bases the safe load capacity on ultimate strength with a safety factor of 1.3, allows even greater load increases at the operating level. The basic load factor relationship for ratings is:

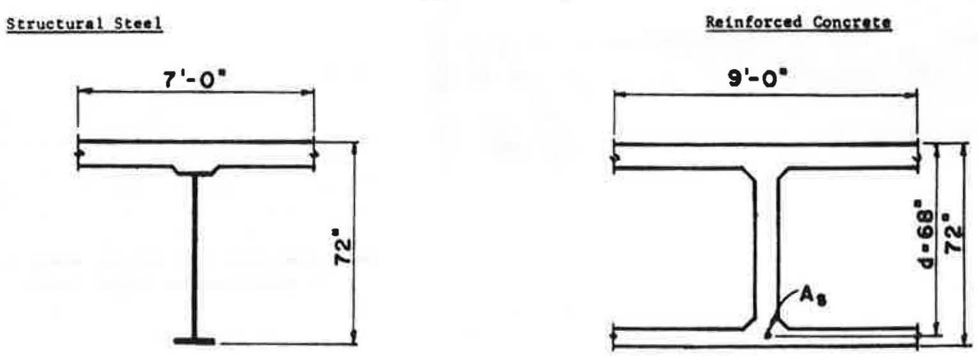
$$\phi \, M_u = 1.3 \left[M_{DL} + M_{(LL+I)} \right] \tag{1}$$

Solving for $M_{(LL+I)}$ gives the live load capacity:

$$M_{(LL+I)} = \frac{\phi \, M_u}{1.3} - M_{DL} \tag{2}$$

This variation in capacity is illustrated in Figure 4 by comparing a steel girder bridge with a reinforced

GIRDER SECTIONS



ASSUMPTIONS

100 ft (30.48 m) simple span	100 ft (30.48 m) simple span
Composite welded plate girder	Reinforced concrete box girder
Girder spacing s = 7'-0" (2.13 m)	Girder spacing s = 9'-0" (2.74 m)
LL distribution $\frac{s}{5.5}$	LL distribution $\frac{s}{7.}$
A-36 structural steel	Grade 60 reinforcement
F_s = 20 ksi (137.9 MPa)	f_s = 24 ksi (165.5 MPa)
F_y = 36 ksi (248.2 MPa)	f_y = 60 ksi (413.7 MPa)
$F_{s(oper)}$ = 27 ksi (186.2 MPa)	$f_{s(oper)}$ = 36 ksi (248.2 MPa)
Concrete: f'_c = 3250 psi (22.41 MPa)	Concrete: f'_c = 3250 psi (22.41 MPa)

Design controlled by maximum positive moments

These assumptions are used in the design examples in Figures 4, 5, and 7

Figure 3

concrete box girder bridge, both having 30.5 meters (100-foot) simple spans. For this example the two structures shown in Figure 3 were selected to have approximately equal LL+I capacity at WSD allowable stresses. At the operating stress level (WSD), however, the concrete structure has nearly 40% more capacity for live load than the steel structure. When compared using the load factor rating method the concrete bridge has nearly twice the live load capacity of the steel bridge.

Load Factor Design

The AASHTO Specifications have allowed the use of LFD as an alternative design method for steel bridges since 1970. Similar provisions were included for reinforced concrete and prestressed concrete soon after that.

There are three fundamental loading conditions used in applying LFD. These are shown in Table 1.

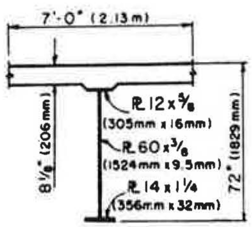
The LFD method, which includes an overload check, produces structures with a more uniform overload capacity. Unfortunately, structures designed by this method using HS20 loads are often not able to accom-

WORKING STRESS

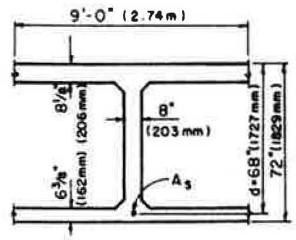
This example represents bridges designed by working stress and demonstrates the method of rating these structures at operating level for both WSD and LFD. (Both are permitted by AASHTO)

GIRDER SECTIONS

Structural Steel



Reinforced Concrete



Working Stress Design

Structural Steel		Reinforced Concrete
0.712 K/ft 10.39 kN/m	Dead Load	2.145 K/ft 31.30 kN/m
890 Kft 1207 kNm	M_{DL}	2681 Kft 3635 kNm
1185 Kft 1607 kNm	$M_{(LL+I)}$	1197 Kft 1623 kNm
2075 Kft 2814 kNm	Design M	3878 Kft 5259 kNm

Structural Steel:

$$\frac{M_{DL}}{S_s} + \frac{M_{(LL+I)}}{S_{comp}} = .55\ F_y$$

$9.4 + 8.9 = 18.3 < 20$ ksi (137.9 MPa)

Reinforced Concrete:

$$M = A_s f_s J d$$

$$A_s = \frac{3878 \times 12}{24(.922)70} = 30\ \text{in}^2\ (193.5\ \text{cm}^2)$$

Live Load Capacity at Operating Level (WSD)

Structural Steel:

$F_{s\text{-}oper} = .75\ F_y = 27$ ksi (186.2 MPa)

$F_{s\text{-}oper} - F_{s\text{-}DL} = F_{s\text{-}(LL+I)oper}$

$27 - 9.4 = F_{s\text{-}(LL+I)oper} = 17.6$ ksi (121.4 MPa)

$M_{(LL+I)oper} = \frac{F_{s(LL+I)oper}}{S_{comp}} = \underline{2335}$ Kft (3166 kNm)

Reinforced Concrete:

$f_{s\text{-}oper} = 36$ ksi (248.2 MPa)

$M_{oper} = (A_s) f_{s\text{-}oper} (J) d = M_{DL} + M_{(LL+I)oper}$

$= \frac{30(36).922(70)}{12} = 5809$ Kft (7877 kNm)

$M_{(LL+I)oper} = M_{oper} - M_{DL}$

$= 5809 - 2681 = \underline{3128}$ Kft (4242 kNm)

Live Load Capacity at Operating Level (LFD)

Structural Steel:

$F_y = 36$ ksi (248.2 MPa) $= M/S$

$$\frac{M_{oper}}{S} = 1.3\left[\frac{M_{DL}}{S_s} + \frac{M_{(LL+I)oper}}{S_{comp}}\right] = 36\ \text{ksi}\ (248.2\ \text{MPa})$$

$\frac{36}{1.3} - 9.4 = 18.3$ ksi (126.2 MPa) $= \frac{M_{(LL+I)oper}}{S_{comp}}$

$M_{(LL+I)oper} = 18.3(S_{comp}) = \underline{2450}$ Kft (3322 kNm)

Reinforced Concrete:

$f_y = 60$ ksi (413.7 MPa)

$M_{oper} = \phi A_s f_y (d - \frac{a}{2}) = 1.3\left[M_{DL} + M_{(LL+I)oper}\right]$

$= \frac{0.9(30)60(66)}{12} = 8910$ Kft (12,082 kNm)

$M_{(LL+I)} = \frac{M_{oper}}{1.3} - M_{DL}$

$= \frac{8910}{1.3} - 2681 = \underline{4173}$ Kft (5656 kNm)

Figure 4

Table 1

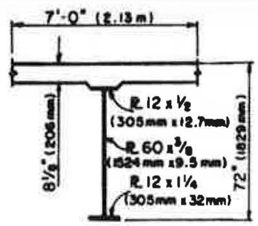
AASHTO
LOAD FACTOR DESIGN

LOADING	PERFORMANCE CONDITION	LIMITING CRITERION: Structural Steel	LIMITING CRITERION: Reinforced Concrete
1. **Service Load** **D+(LL+I)**	LL Deflection	L/Factor	L/Factor
	Fatigue	Controls on stress range same as for WSD	Controls on stress range in reinforcement. Concrete-$0.5f_c'$ (Reversal Areas)
	Concrete Crack Control	None	Limits on stress in reinforcement based on cover and spacing
2. Overload D+5/3(LL+I)	Prevent excessive permanent distortion under an occasional overload	Noncomposite: $F_s = 0.8\ F_yS$ Composite: $F_s = 0.95\ F_yS$	None
3. Max Design Load 1.3[D+5/3(LL+I)]	Provide a reasonable factor of safety against collapse	Noncompact section $M_u = F_yS$ Compact Section $M_u = F_yZ$	All compression in flange $M = \emptyset A_s f_y (d-a/2)$ Compression in flange & web $M = \emptyset(A_s-A_{sf})f_y(d-a/2) + A_{sf}f_y(d-0.5h_f)$

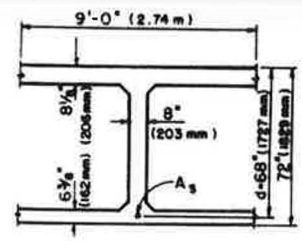
AASHTO LOAD FACTOR

These girders have been designed and rated using Load Factor Design procedures shown in AASHTO.

Structural Steel



Reinforced Concrete



Load Factor Design

Structural Steel		Reinforced Concrete
890 Kft **1207 kNm**	M_{DL}	**2681 Kft** **3635 kNm**
1185 Kft **1607 kNm**	$M_{(LL+I)}$	**1197 Kft** **1623 kNm**
3725 Kft **5051 kNm**	$M_u = 1.3\left[M_{DL} + 5/3\ M_{(LL+I)}\right] = F_yS$	**6079 Kft** **8247 kNm**
	Overload (M_o)	
2865 Kft **3885 kNm** **(77% of M_u)**	$M_o = M_{DL} + 5/3\ M_{(LL+I)} = 0.8\ F_yS$	**4676 Kft** **6341 kNm** **(77% of M_u)**

By inspection Overload does not control

Live Load Capacity at Operating Level

Structural Steel		Reinforced Concrete
3725 Kft **5051 kNm**	$M_u = 1.3\left[M_{DL} + M_{(LL+I)oper}\right]$	**6079 Kft** **8247 kNm**

$$M_{(LL+I)oper} = \frac{M_u}{1.3} - M_{DL}$$

$$M_{(LL+I)oper} = \frac{3725}{1.3} - 890$$

$$= \underline{1974}\ \text{Kft}$$

(2677 kNm)

$$M_{(LL+I)oper} = \frac{6079}{1.3} - 2681$$

$$= \underline{1995}\ \text{Kft}$$

(2705 kNm)

Figure 5

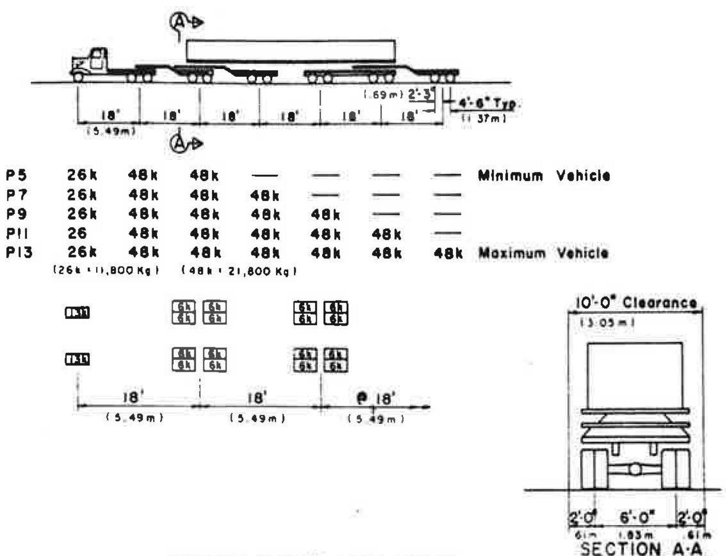
modate permit loads that have been carried safely on older structures designed by WSD.

This can be illustrated by going back to the bridge used in the earlier example. Figure 5 shows a comparison of the capacity for live load at operating stress level for a similar steel and concrete structure designed by LFD using HS20 live loads.

With this design method the capacities for live load using the load factor rating system are nearly equal, i.e., both structures have the same capacity for permit loads. The problem is that this capacity is significantly reduced under those for structures designed by WSD. Randomly located bridges designed by these criteria can severely limit the usability of a highway network.

Need for New Criteria

Perhaps it was good fortune that specifications for LFD and the federal requirements for inventorying and rating existing structures occurred at about the same time. This forced us to consider bridge design methods in relation to the way we intended to utilize the completed structures. It became apparent that when our Maintenance Engineer added a new structure



PERMIT DESIGN LIVE LOADS

Figure 6

Table 2

CALIFORNIA CURRENT PRACTICE

LOADING	PERFORMANCE CONDITION	LIMITING CRITERION: Structural Steel	LIMITING CRITERION: Reinforced Concrete
1. Service Loads $D+(LL+I)_H$	LL Deflection	L/Factor	L/Factor
	Fatigue	Load repetitions and stress ranges defined by AASHTO	Controls on stress range in reinforcement. Concrete-$0.5f'_c$ (Reversal Areas)
	Concrete Crack Control	None	Limits on stress in reinforcement based on cover and spacing
$D+(LL+I)_P$	Fatigue	100,000 cycles - Stress ranges defined by AASHTO	None
2. Max Design Load $1.3[D+5/3(LL+I)_H]$	Provide a reasonable factor of safety against collapse using HS20 loads[a]	Noncomposite section $M_u = F_yS$	All compression in flange $M = \phi A_s f_y(d-a/2)$
		Compact section $M_u = F_yZ$	Compression in flange & web $M = \phi\ (A_s-A_{sf})f_y(d-a/2) + A_{sf}f_y(d-0.5h_f)$
3. Max Design Load $1.3[D+(LL+I)_P]$	Provide a reasonable factor of safety against collapse using the permit vehicle (P-Truck)[a]	Noncomposite section $M_u = F_yS$	All compression in flange $M = \phi A_s f_y(d-a/2)$
		Compact section $M_u = F_yZ$	Compression in flange & web $M = \phi\ (A_s-A_{sf})f_y(d-a/2) + A_{sf}f_y(d-0.5h_f)$

a - Values from the envelope of the maximum effects of the HS or P loadings, whichever controls at a given section, are used to proportion members.

to his inventory, he immediately reanalyzed it to rate it for extra-legal loads. Under fresh scrutiny, the practice of designing new bridges by one criteria and rating them by another seemed a little absurd.

We saw that the continued use of WSD, which results in a haphazard array of capacities at the operating stress level had some obvious disadvantages. A few structures with low overload capacities limited the usability of entire routes. Other structures on the same route had far greater overload capacity than we could reasonably expect to use.

The adoption of LFD with only HS20 loadings was out of the question for us. We could not bear the cries of anguish that would be forthcoming from the truckers if the level of permit loads was suddenly lowered. Besides, it seemed foolish to knowingly downgrade our highway system in the face of continuing pressure to increase both legal and permit loads.

We gave serious consideration to adopting LFD with a larger HS vehicle, perhaps an HS25 or HS30. This was rejected in favor of designing each new structure for both a standard permit vehicle and HS20 loadings. We eventually realized that there was only one way to directly correlate design practice with permit policy: Design the structure for the load you expect to put on it using operating stress levels. In other words attaining the desired permit load capacity is made one of the performance conditions in the design procedure.

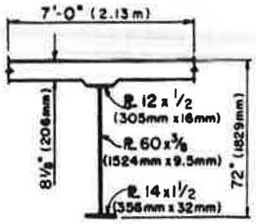
Permit Vehicle

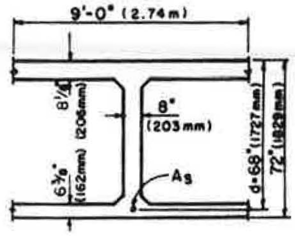
To provide a systematic method of rating bridges for permit purposes, California bridge maintenance engineers devised a loading system that has axle weights approximately 1.5 times legal loads. As shown in Figure 6, this family of overload vehicles, which we call "P-loads", is a set of five trucks. Each of the five is composed of a steering axle and from two to six pairs of tandem axles at 5.49 meters (18-foot) centers. The total length ranges from 10.98 meters (36 feet) to 32.94 meters (108 feet). The width is 2.44 meters (8 feet) with the wheel lines 1.83 meters (6 feet) apart. The heaviest of the series, the P-13 loading, has a gross weight of

CALTRANS LOAD FACTOR

These girders have been designed according to Caltrans' LFD procedures and rated using AASHTO LFD criteria.

Structural Steel | **Reinforced Concrete**





Load Factor Design
(Caltrans)

Structural Steel		Reinforced Concrete
890 Kft 1207 kNm	M_{DL}	2681 Kft 3635 kNm
1185 Kft 1607 kNm	$M_{(LL+I)H}$	1197 Kft 1623 kNm
2652 Kft 3596 kNm	$M_{(LL+I)P}$	2679 Kft 3633 kNm
3724 Kft 5050 kNm	$M_u = 1.3\left[M_{DL} + 5/3\ M_{(LL+I)H}\right]$	6079 Kft 8247 kNm
	or	
4605 Kft 6244 kNm	$1.3\left[M_{DL} + M_{(LL+I)P}\right]$	6968 Kft 9449 kNm

Live Load Capacity at Operating Level

4605 Kft 6244 kNm	$M_{oper} = 1.3\left[M_{DL} + M_{(LL+I)oper}\right] = M_u$	6968 Kft 9449 kNm

$$M_{(LL+I)oper} = \frac{M_{oper}}{1.3} - M_{DL}$$

$$M_{(LL+I)oper} = \frac{4605}{1.3} - 890 = \underline{\underline{2652}}\ \text{Kft}\ (3596\ \text{kNm})$$

$$M_{(LL+I)oper} = \frac{6968}{1.3} - 2681 = \underline{\underline{2679}}\ \text{Kft}\ (3633\ \text{kNm})$$

Figure 7

142,242 kg (314,000 pounds).

On many routes, routine permits are issued for vehicles closely resembling the P-9 truck. The P-13 truck, which is larger than anything that has actually travelled California's highways with a routine permit, was chosen as an upper limit to allow a reasonable margin for growth. While this load seems immense compared with an HS20 vehicle, existing structures with a high D/(LL+I) ratio that were designed by WSD can carry the P-13 truck without exceeding operating stresses.

Current Practice

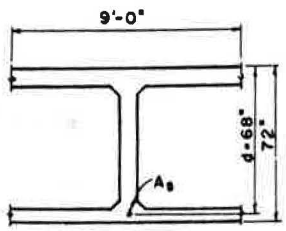
Our current design practice is an adaptation of the AASHTO LFD specifications using P-loads in addition to HS20 loads. All loading combinations except Group 1 come directly from AASHTO and are applied using HS20 loading. Group 1 has been expanded by adding another loading condition using the family of P-loads. The AASHTO overload check with an HS20 truck is not made because it never controls. The loading and limiting criteria are illustrated in Table 2.

For narrow girder spacings California uses AASHTO "S-over" factors for distributing P loads to individual girders. For wider girder spacings one P vehicle and one HS vehicle are placed in adjacent lanes and the distribution to individual girders is found by assuming simple beam action on the slab. This method of distribution is conservative, and we hope eventually that better distribution factors will be developed.

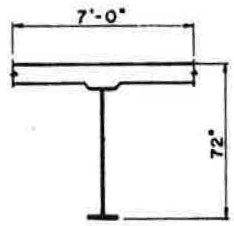
Figure 7 shows the results of a redesign of the same two structures using California's current

LIVE LOAD CAPACITY
(Operating Level)

Reinforced Concrete



Structural Steel



Design Load	LL Capacity	Criteria	LL Capacity
HS20	2335 Kft 3166 kNm	W. S. Design W. S. Rating	3128 Kft 4242 kNm
HS20	2450 Kft 3322 kNm	W. S. Design L. F. Rating	4173 Kft 5656 kNm
HS20	1974 Kft 2677 kNm	L. F. Design (HS) L. F. Rating	1995 Kft 2705 kNm
HS20 & P-Loads	2652 Kft 3596 kNm	L. F. Design L. F. Rating	2679 Kft 3633 kNm

(HS) - AASHTO Live Load
(P) - Caltrans Permit Live Loads

Figure 8

DEFINITIONS AND NOTATIONS

- a Depth of equivalent rectangular stress block for balanced strain conditions
- A_s Area of reinforcing steel
- A_{sf} Area of reinforcement to develop compressive strength of overhanging flanges
- f'_c Specified 28 day compressive strength of concrete
- f_s Allowable stress in reinforcing steel
- F_s Allowable stress in structural steel
- f_y Specified yield strength of reinforcement
- F_y Specified minimum yield strength of structural steel
- h_f Compression flange thickness of I and T sections
- L Span length
- LFD Load Factor Design
- $(LL+I)_H$ Live Load effects of H (M) loads
- $(LL+I)_P$ Live Load effects of P loads
- M_{DL} Moment due to dead load
- $M_{(LL+I)}$ Moment due to live load + impact
- M_o Overload Moment
- M_u Maximum moment capacity
- P-Loads Family of truck loads used in California Permit Policy
- S Section modulus
- S_{comp} Section Modulus of composite structural steel
- S_s Section Modulus of structural steel only
- ϕ Strength Reduction Factor
- WSD Working Stress Design
- Z Plastic Section Modulus

practice. This procedure produces structures of equal capacity for permit loads, and this capacity is greatly increased over that obtained using LFD and HS20 loads.

Live load capacities for permit loads obtained using the three design methods under consideration are summarized in Figure 8. The notable general characteristics of bridges designed using California's current practice are:

- Live load capacity for permit loads is essentially equal for all structure types.
- This capacity is higher than that which would result from using LFD with HS20 loads only.
- This capacity may be more or less than the wide array of values obtained using WSD methods.

While the criteria listed in Table 2 are simple and straightforward, implementing this change in procedure has not been easy. Computer programs that were based on HS20 loads had to be modified to generate moments and shears for P-loads. Designers had to be indoctrinated in LFD concepts and learn many complex rules included in the AASHTO Specifications. A forty-year collection of charts, design aids and short-cuts based on WSD became obsolete instantaneously. Training, developing new computer software, and general wheel-spinning raised engineering costs and slowed production temporarily.

Effect on Structure Types

Using LFD and our permit vehicle, we have found that reinforced concrete bridges require as much as 20% less reinforcing steel than those designed by WSD. For prestressed concrete structures we have found it necessary to either add significant amounts of mild steel to satisfy the ultimate moments caused by the overload vehicle in the common span ranges or use a higher prestress force. For structural steel bridges, slightly heavier steel sections are usually needed to accommodate our overload vehicle. In some cases, the steel section is controlled by fatigue considerations rather than the ultimate load capacity.

Based on our typical mix of structures, the total cost of our bridges has not changed significantly as a result of this design procedure.

Conclusions

The correlation of design method with permit policy was long overdue in California. It was slow in coming because nobody looked at the big picture. The designer followed AASHTO WSD rules using HS20 loads, which is common practice in the majority of states. The maintenance engineer was apparently content to live with these new designs even though his evaluations showed a wide disparity in their capacity for loads at operating stress levels.

It took consideration of implementing LFD to start a meaningful dialogue between the designer and the maintenance engineer. Both agreed that a general decrease in load capacity was unreasonable. Both thought that providing a uniform load capacity at the operating stress level was a desirable goal. It was out of these discussions that California's current design practice evolved. The advantage is considerable. For no increase in cost we are building new bridges that have a uniform, planned capacity to carry standard permit loads at the operating stress level. Being able to fully utilize every structure along a route provides maximum usability at least cost.

Before starting on an accelerated bridge replacement program, bridge engineers should review their design methods as they relate to policies concerning routine permit loads. They must decide on the stress level at which they want uniform capacity in their bridges. Particularly, they should take a hard look at the kind of structures they will get using LFD with HS20 loads only. The decisions that are made concerning design criteria will have a significant impact on the usability of the transportation network. The importance of studying this issue carefully cannot be overemphasized.