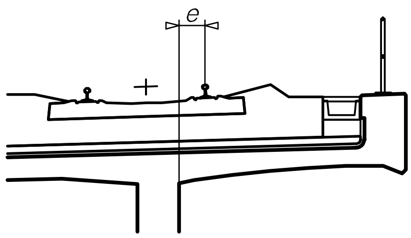
Table 6.2 (continued)

Case	Structural element	Determinant length L_Φ
Concrete deck slab with ballast bed (for local and transverse stresses)		
4.1	Deck slab as part of box girder or upper flange of main beam - spanning transversely to the main girders - spanning in the longitudinal direction - cross girders - transverse cantilevers supporting railway loading	 3 times span of deck plate 3 times span of deck plate Twice the length of the cross girder  - $e \leq 0,5$ m: 3 times the distance between the webs - $e > 0,5$ m: [a] **Figure 6.11 - Transverse cantilever supporting railway loading**
4.2	Deck slab continuous (in main girder direction) over cross girders	Twice the cross girder spacing
4.3	Deck slab for half through and trough bridges: - spanning perpendicular to the main girders - spanning in the longitudinal direction	 Twice span of deck slab + 3m Twice span of deck slab
4.4	Deck slabs spanning transversely between longitudinal steel beams in filler beam decks	Twice the determinant length in the longitudinal direction
4.5	Longitudinal cantilevers of deck slab	- $e \leq 0,5$ m: 3,6m [b] - $e > 0,5$ m: [a]
4.6	End cross girders or trimmer beams	3,6m [b]

[a] In general all cantilevers greater than 0,50 m supporting rail traffic actions need a special study in accordance with 6.4.6 and with the loading agreed with the relevant authority specified in the National Annex.
[b] It is recommended to apply Φ_3

NOTE For Cases 1.1 to 4.6 inclusive L_Φ is subject to a maximum of the determinant length of the main girders.

Table 6.2 (continued)

Case	Structural element	Determinant length L_Φ
Main girders		
5.1	Simply supported girders and slabs (including steel beams embedded in concrete)	Span in main girder direction
5.2	Girders and slabs continuous over n spans with $L_m = 1/n\,(L_1 + L_2 + .. + L_n)$ (6.6)	$L_\Phi = k \times L_m$, (6.7) but not less than max L_i ($i = 1,..., n$) $n =$ 2 3 4 ≥ 5 $k =$ 1,2 1,3 1,4 1,5
5.3	Portal frames and closed frames or boxes:	
	- single-span	Consider as three-span continuous beam (use 5.2, with vertical and horizontal lengths of members of the frame or box)
	- multi-span	Consider as multi-span continuous beam (use 5.2, with lengths of end vertical members and horizontal members)
5.4	Single arch, archrib, stiffened girders of bowstrings	Half span
5.5	Series of arches with solid spandrels retaining fill	Twice the clear opening
5.6	Suspension bars (in conjunction with stiffening girders)	4 times the longitudinal spacing of the suspension bars
Structural supports		
6	Columns, trestles, bearings, uplift bearings, tension anchors and for the calculation of contact pressures under bearings.	Determinant length of the supported members

6.4.5.4 Reduced dynamic effects

(1) In the case of arch bridges and concrete bridges of all types with a cover of more than 1,00 m, Φ_2 and Φ_3 may be reduced as follows:

$$red\,\Phi_{2,3} = \Phi_{2,3} - \frac{h - 1{,}00}{10} \geq 1{,}0 \qquad (6.8)$$

where:

h is the height of cover including the ballast from the top of the deck to the top of the sleeper, (for arch bridges, from the crown of the extrados) [m].

(2) The effects of rail traffic actions on columns with a slenderness (buckling length/radius of gyration) < 30, abutments, foundations, retaining walls and ground pressures may be calculated without taking into account dynamic effects.

6.4.6 Requirements for a dynamic analysis

6.4.6.1 Loading and load combinations

6.4.6.1.1 Loading

(1)P The dynamic analysis shall be undertaken using characteristic values of the loading from the Real Trains specified. The selection of Real Trains shall take into account each permitted or envisaged train formation for every type of high speed train permitted or envisaged to use the structure at speeds over 200km/h.

NOTE 1 The individual project may specify the characteristic axle loads and spacings for each configuration of each required Real Train.

NOTE 2 Also see 6.4.6.1.1(7) for loading where a dynamic analysis is required for a Maximum Line Speed at the Site less than 200km/h.

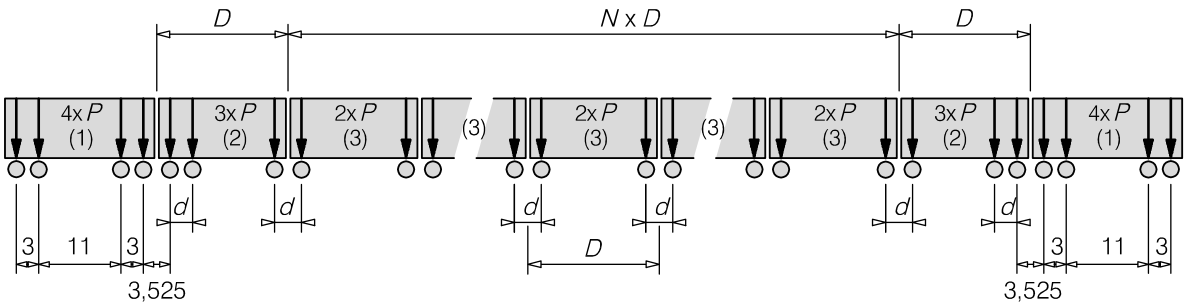
(2)P The dynamic analysis shall also be undertaken using Load Model HSLM on bridges designed for international lines where European high speed interoperability criteria are applicable.

NOTE The individual project may specify when Load Model HSLM is to be used.

(3) Load Model HSLM comprises of two separate Universal Trains with variable coach lengths, HSLM-A and HSLM-B.

NOTE HSLM-A and HSLM-B together represent the dynamic load effects of articulated, conventional and regular high speed passenger trains in accordance with the requirements for the European Technical Specification for Interoperability given in E.1.

(4) HSLM-A is defined in Figure 6.12 and Table 6.3:



Key

(1) Power car (leading and trailing power cars identical)

(2) End coach (leading and trailing end coaches identical)

(3) Intermediate coach

Figure 6.12 - HSLM-A

Table 6.3 - HSLM-A

Universal Train	Number of intermediate coaches N	Coach length D [m]	Bogie axle spacing d [m]	Point force P [kN]
A1	18	18	2,0	170
A2	17	19	3,5	200
A3	16	20	2,0	180
A4	15	21	3,0	190
A5	14	22	2,0	170
A6	13	23	2,0	180
A7	13	24	2,0	190
A8	12	25	2,5	190
A9	11	26	2,0	210
A10	11	27	2,0	210

(5) HSLM-B comprises of N number point forces of 170 kN at uniform spacing d [m] where N and d are defined in Figures 6.13 and 6.14:

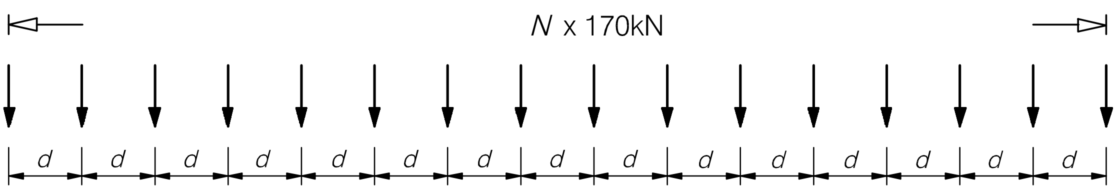


Figure 6.13 - HSLM-B

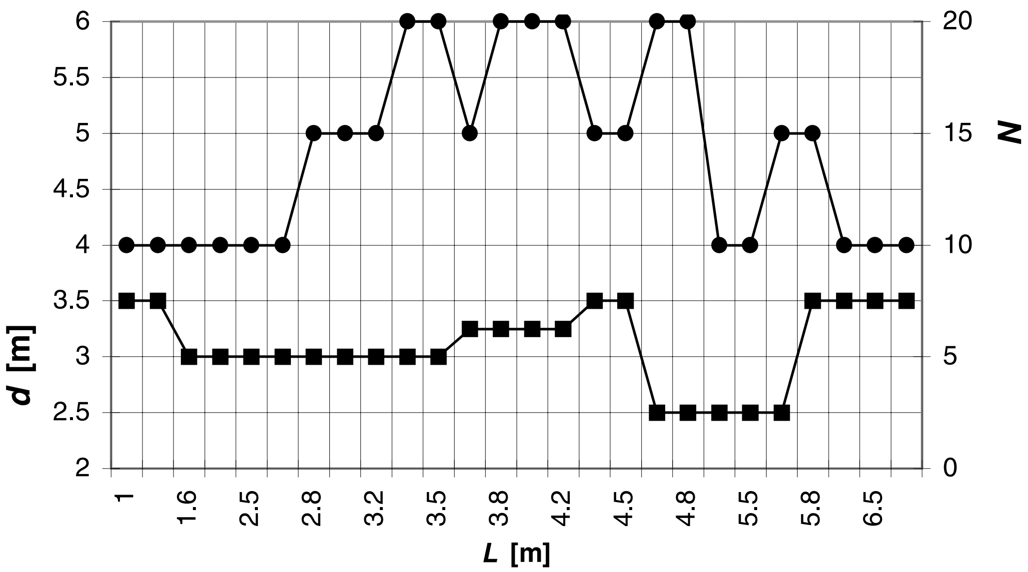


Figure 6.14 - HSLM-B

where L is the span length [m].

(6) Either HSLM-A or HSLM-B should be applied in accordance with the requirements of Table 6.4:

Table 6.4 - Application of HSLM-A and HSLM-B

Structural configuration	Span	
	$L < 7$m	$L \geq 7$m
Simply supported span[a]	HSLM-B[b]	HSLM-A[c]
Continuous structure[a] or Complex structure[e]	HSLM-A Trains A1 to A10 inclusive[d]	HSLM-A Trains A1 to A10 inclusive[d]

[a] Valid for bridges with only longitudinal line beam or simple plate behaviour with negligible skew effects on rigid supports.
[b] For simply supported spans with a span of up to 7 m a single critical Universal Train from HSLM-B may be used for the analysis in accordance with 6.4.6.1.1(5).
[c] For simply supported spans with a span of 7 m or greater a single critical Universal Train from HSLM-A may be used for the dynamic analysis in accordance with annex E (Alternatively Universal trains A1 to A10 inclusive may be used).
[d] All Trains A1 to A10 inclusive should be used in the design.
[e] Any structure that does not comply with Note a above. For example a skew structure, bridge with significant torsional behaviour, half through structure with significant floor and main girder vibration modes etc. In addition, for complex structures with significant floor vibration modes (*e.g.* half through or through bridges with shallow floors) HSLM-B should also be applied.

NOTE The National Annex or the individual project may specify additional requirements relating to the application of HSLM-A and HSLM-B to continuous and complex structures.

(7) Where the frequency limits of Figure 6.10 are not satisfied and the Maximum Line Speed at the Site is ≤ 200 km/h a dynamic analysis should be carried out. The analysis should take into account the behaviours identified in 6.4.2 and consider:

– Train Types 1 to 12 given in annex D,
– Real Trains specified.

NOTE The loading and methodology for the analysis may be specified for the individual project and should be agreed with the relevant authority specified in the National Annex.

6.4.6.1.2 Load combinations and partial factors

(1) For the dynamic analysis the calculation of the value of mass associated with self weight and removable loads (ballast etc.) should use nominal values of density.

(2)P For the dynamic analysis loads according to 6.4.6.1.1(1) and (2) and where required 6.4.6.1.1(7) shall be used.

(3) For the dynamic analysis of the structure only, one track (the most adverse) on the structure should be loaded in accordance with Table 6.5.

Table 6.5 - Summary of additional load cases depending upon number of tracks on bridge

Number of tracks on a bridge	Loaded track	Loading for dynamic analysis
1	one	Each Real Train and Load Model HSLM (if required) travelling in the permitted direction(s) of travel.
2 (Trains normally travelling in opposite directions) [a]	either track	Each Real Train and Load Model HSLM (if required) travelling in the permitted direction(s) of travel.
	other track	None.

[a] For bridges carrying 2 tracks with trains normally travelling in the same directions or carrying 3 or more tracks with a Maximum Line Speed at the Site exceeding 200km/h the loading should be agreed with the relevant authority specified in the National Annex.

(4) Where the load effects from a dynamic analysis exceed the effects from Load Model 71 (and Load Model SW/0 for continuous structures) in accordance with 6.4.6.5(3) on a track the load effects from a dynamic analysis should be combined with:
– the load effects from horizontal forces on the track subject to the loading in the dynamic analysis,
– the load effects from vertical and horizontal loading on the other track(s), in accordance with the requirements of 6.8.1 and Table 6.11.

(5)P Where the load effects from a dynamic analysis exceed the effects from Load Model 71 (and Load Model SW/0 for continuous structures) in accordance with 6.4.6.5(3) the dynamic rail loading effects (bending moments, shears, deformations etc. excluding acceleration) determined from the dynamic analysis shall be enhanced by the partial factors given in A2 of EN 1990.

(6)P Partial factors shall not be applied to the loading given in 6.4.6.1.1 when determining bridge deck accelerations. The calculated values of acceleration shall be directly compared with the design values in 6.4.6.5.

(7) For fatigue, a bridge should be designed for the additional fatigue effects at resonance from the loading in accordance with 6.4.6.1.1 on any one track. See 6.4.6.6.

6.4.6.2 Speeds to be considered

(1)P For each Real Train and Load Model HSLM a series of speeds up to the Maximum Design Speed shall be considered. The Maximum Design Speed shall be generally 1,2 × Maximum Line Speed at the site.

The Maximum Line Speed at the site shall be specified.

NOTE 1 The individual project may specify the Maximum Line Speed at the site.

NOTE 2 Where specified for the individual project a reduced speed may be used for checking individual Real Trains for 1,2 × their associated Maximum Permitted Vehicle Speed.

NOTE 3 It is recommended that the individual project specify an increased Maximum Line Speed at the Site to take into account potential modifications to the infrastructure and future rolling stock.

NOTE 4 Structures can exhibit a highly peaked response due to resonance effects. Where there is a likelihood of train overspeeding and exceeding either the Maximum Permitted Vehicle Speed or the current or envisaged Maximum Line Speed at the Site it is recommended that the individual project specify an additional factor for increasing the Maximum Design Speed to be used in the dynamic analysis.

NOTE 5 It is recommended that the individual project specify additional requirements for checking structures where there is a requirement for a section of line to be suitable for commissioning tests of a Real Train. The Maximum Design Speed used for the Real Train should be at least 1,2 × Maximum Train Commissioning Speed. Calculations are required to demonstrate that safety considerations (maximum deck accelerations, maximum load effects, etc.) are satisfactory for structures at speeds in excess of 200 km/h. Fatigue and passenger comfort criteria need not be checked at 1,2 × Maximum Train Commissioning Speeds.

(2) Calculations should be made for a series of speeds from 40m/s up to the Maximum Design Speed defined by 6.4.6.2(1). Smaller speed steps should be made in the vicinity of Resonant Speeds.

For simply supported bridges that may be modelled as a line beam the Resonant Speeds may be estimated using Equation 6.9.

$$v_{\mathrm{i}} = n_0 \lambda_{\mathrm{i}} \qquad (6.9)$$

and

$$40 \text{ m/s} \leq v_i \leq \text{Maximum Design Speed,} \qquad (6.10)$$

where:

v_i is the Resonant Speed [m/sec]
n_0 is the first natural frequency of the unloaded structure,
λ_i is the principal wavelength of frequency of excitation and may be estimated by:

$$\lambda_i = \frac{d}{i} \qquad (6.11)$$

d is the regular spacing of groups of axles
i = 1, 2, 3 or 4.

6.4.6.3 Bridge parameters

6.4.6.3.1 Structural damping

(1) The peak response of a structure at traffic speeds corresponding to resonant loading is highly dependent upon damping.

(2)P Only lower bound estimates of damping shall be used.

(3) The following values of damping should be used in the dynamic analysis:

Table 6.6 - Values of damping to be assumed for design purposes

Bridge Type	ζ Lower limit of percentage of critical damping [%]	
	Span L < 20m	Span $L \geq$ 20m
Steel and composite	$\zeta = 0{,}5 + 0{,}125\,(20 - L)$	$\zeta = 0{,}5$
Prestressed concrete	$\zeta = 1{,}0 + 0{,}07\,(20 - L)$	$\zeta = 1{,}0$
Filler beam and reinforced concrete	$\zeta = 1{,}5 + 0{,}07\,(20 - L)$	$\zeta = 1{,}5$

NOTE Alternative safe lower bound values may be used subject to the agreement of the relevant authority specified in the National Annex.

6.4.6.3.2 Mass of the bridge

(1) Maximum dynamic load effects are likely to occur at resonant peaks when a multiple of the frequency of loading and a natural frequency of the structure coincide and any underestimation of mass will overestimate the natural frequency of the structure and overestimate the traffic speeds at which resonance occurs.

At resonance the maximum acceleration of a structure is inversely proportional to the mass of the structure.

(2)P Two specific cases for the mass of the structure including ballast and track shall be considered:

– a lower bound estimate of mass to predict maximum deck accelerations using the minimum likely dry clean density and minimum thickness of ballast,
– an upper bound estimate of mass to predict the lowest speeds at which resonant effects are likely to occur using the maximum saturated density of dirty ballast with allowance for future track lifts.

NOTE The minimum density of ballast may be taken as 1700kg/ m^3. Alternative values may be specified for the individual project.

(3) In the absence of specific test data the values for the density of materials should be taken from EN 1991-1-1.

NOTE Owing to the large number of parameters which can affect the density of concrete it is not possible to predict enhanced density values with sufficient accuracy for predicting the dynamic response of a bridge. Alternative density values may be used when the results are confirmed by trial mixes and the testing of samples taken from site in accordance with EN 1990, EN 1992 and ISO 6784 subject to the agreement of the relevant authority specified in the National Annex.

6.4.6.3.3 Stiffness of the bridge

(1) Maximum dynamic load effects are likely to occur at resonant peaks when a multiple of the frequency of loading and a natural frequency of the structure coincide. Any overestimation of bridge stiffness will overestimate the natural frequency of the structure and speed at which resonance occurs.

(2)P A lower bound estimate of the stiffness throughout the structure shall be used.

(3) The stiffness of the whole structure including the determination of the stiffness of elements of the structure may be determined in accordance with EN 1992 to EN 1994.

Values of Young's modulus may be taken from EN 1992 to EN 1994.

For concrete compressive cylinder strength $f_{ck} \geq 50$ N/mm^2 (compressive cube strength $f_{ck,\ cube} \geq 60$ N/mm^2) the value of static Young's modulus (E_{cm}) should be limited to the value corresponding to a concrete of strength of $f_{ck} = 50$ N/mm^2 ($f_{ck,\ cube} = 60$ N/mm^2).

NOTE 1 Owing to the large number of parameters which can affect E_{cm} it is not possible to predict enhanced Young's modulus values with sufficient accuracy for predicting the dynamic response of a bridge. Enhanced E_{cm} values may be used when the results are confirmed by trial mixes and the testing of samples taken from site in accordance with EN 1990, EN 1992 and ISO 6784 subject to the agreement of the relevant authority specified in the National Annex.

NOTE 2 Other material properties may be used subject to the agreement of the relevant authority specified in the National Annex.

6.4.6.4 Modelling the excitation and dynamic behaviour of the structure

(1) The dynamic effects of a Real Train may be represented by a series of travelling point forces. Vehicle/structure mass interaction effects may be neglected.

The analysis should take into account variations throughout the length of the train in axle forces and the variations in spacing of individual axles or groups of axles.

(2) Where appropriate the analysis technique should allow for the following dynamic behaviours of the structure:

– for complex structures the proximity of adjacent frequencies and associated mode shapes of oscillation,
– interaction between bending and torsional modes,
– local deck element behaviour (shallow floors and cross girders of half-through type bridges or trusses etc.),
– the skew behaviour of slabs etc.

(3) The representation of each axle by a single point force tends to overestimate dynamic effects for loaded lengths of less than 10m. In such cases, the load distribution effects of rails, sleepers and ballast may be taken into account.

Notwithstanding 6.3.6.2(1) individual axle loads should not be distributed uniformly in the longitudinal direction for a dynamic analysis.

(4) For spans less than 30 m dynamic vehicle/bridge mass interaction effects tend to reduce the peak response at resonance. Account may be taken of these effects by:

– carrying out a dynamic vehicle/structure interactive analysis,

NOTE The method used should be agreed with the relevant authority specified in the National Annex.

– increasing the value of damping assumed for the structure according to Figure 6.15. For continuous beams, the smallest value $\Delta\zeta$ for all spans should be used. The total damping to be used is given by :

$$\zeta_{TOTAL} = \zeta + \Delta\zeta \qquad (6.12)$$

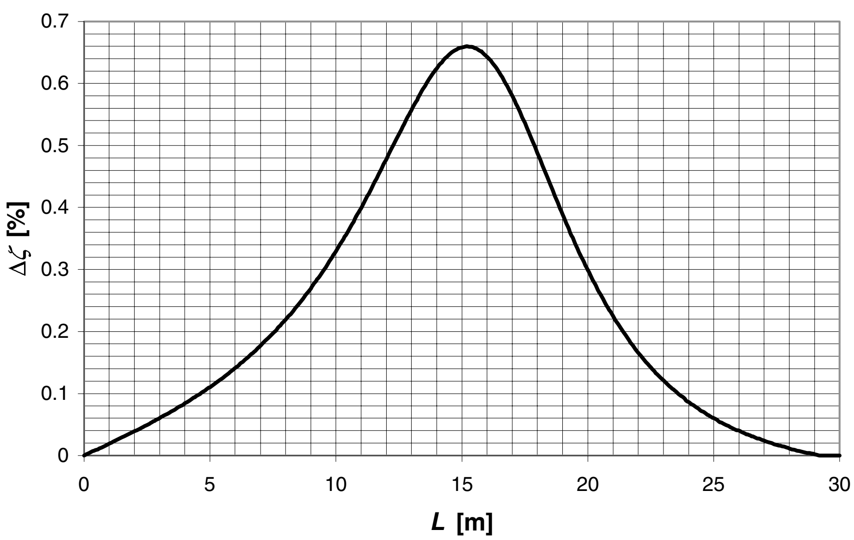


Figure 6.15 - Additional damping $\Delta\zeta$ [%] as a function of span length L [m]

where:

$$\Delta\zeta = \frac{0{,}0187L - 0{,}00064L^2}{1 - 0{,}0441L - 0{,}0044L^2 + 0{,}000255L^3}[\%] \qquad (6.13)$$

ζ is the lower limit of percentage of critical damping [%] defined in 6.4.6.3.1.

NOTE The National Annex may specify alternative values.

(5) The increase in calculated dynamic load effects (stresses, deflections, bridge deck accelerations, etc.) due to track defects and vehicle imperfections may be estimated by multiplying the calculated effects by a factor of:
$(1 + \varphi''/2)$ for carefully maintained track,
$(1 + \varphi'')$ for track with standard track maintenance,

where:

φ'' is in accordance with annex C and should not be taken as less than zero.

NOTE The National Annex may specify the factor to be used.

(6) Where the bridge satisfies the upper limit in Figure 6.10 the factors that influence dynamic behaviours (vii) to (xi) identified in 6.4.2 may be considered to be allowed for in Φ, $\varphi''/2$ and φ'' given in 6.4 and annex C.

6.4.6.5 Verifications of the limit states

(1)P To ensure traffic safety:
- The verification of maximum peak deck acceleration shall be regarded as a traffic safety requirement checked at the serviceability limit state for the prevention of track instability.
- The dynamic enhancement of load effects shall be allowed for by multiplying the static loading by the dynamic factor Φ defined in 6.4.5. If a dynamic analysis is necessary, the results of the dynamic analysis shall be compared with the results of the static analysis enhanced by Φ (and if required multiplied by α in accordance with 6.3.2) and the most unfavourable load effects shall be used for the bridge design.
- If a dynamic analysis is necessary, a check shall be carried out according to 6.4.6.6 to establish whether the additional fatigue loading at high speeds and at resonance is covered by consideration of the stresses due to load effects from Φ x LM71 (and if required $\Phi \times$ Load Model SW/0 for continuous structures and classified vertical load in accordance with 6.3.2(3) where required). The most adverse fatigue loading shall be used in the design.

(2)P The maximum permitted peak design values of bridge deck acceleration calculated along the line of a track shall not exceed the recommended values given in A2 of EN 1990 (see A2.4.4.2.1).

(3) A dynamic analysis (if required) should be used to determine the following dynamic enhancement :

$$\varphi'_{dyn} = \max \left| y_{dyn} / y_{stat} \right| - 1 \qquad (6.14)$$

where: